Adelaide Coast Protection Strategy Review 1984

Prepared for

The Coast Protection Board

South Australia

by

The Coastal Management Branch

Conservation Programmes Division

Department of Environment and Planning

South Australia

March 1984

Insofar as this report carries recommendations for future action, it must not be assumed from the report that they form part of any approved policy.

STUDY TEAM

Wynne, A. A. (Project Leader)

Fotheringham, D. G.

Freeman, R.

Moulds, B.

Penney, S. W.

Petrusevics, P.

Tucker, R.

Ellis, D. P. (Branch Manager)

SUPPORTING CONSULTANT STUDIES

Adelaide Coast Protection Alternatives Study. By Kinhill Stearns and Riedel & Byrne Pty Ltd, April 1983.

Replenishment of Adelaide Beaches from Offshore Sard Sources – Feasibility Study. By Lange, Dames and Campbell Aust. Pty Ltd, and Pickands, Mather and Co.

FOREWORD

The value of the metropolitan Adelaide beaches for recreation and tourism is well recognised but their importance for the continued protection of seafront roads and property is less well understood. The beaches are subject to ongoing erosion processes and cannot be taken for granted. The early loss of coastal dunes for urban development means that measures continue to be needed if the beaches are to be retained for all to enjoy and if costly damage to public and private property is to be avoided.

This report provides a timely review of knowledge about the Adelaide coastal processes and also an updated assessment of the measures available to protect the coast. It is now 12 years since the Coast Protection Board was established and 14 years since the last major report on protecting the Adelaide coast. This previous report, prepared by the University of Adelaide, led to the rock seawall/beach replenishment strategy, which has been implemented over the past 12 years, partly by the State Government, through the Board, and partly by local councils with Government assistance.

The present report reviews all current information and compares in detail all practical alternative strategies, including that which is presently employed.

The report, of necessity, adopts a scientific and engineering approach, and the level of detail makes it particularly appropriate for use by the State Government and seaside councils. However, I am sure that it will be read with interest by all those who would like to know more about the behaviour of the Adelaide beaches. Those parts describing the coastal processes may also prove useful to schools and universities.

I congratulate the Coast Protection Board and the Coastal Management Branch on this study, and have much pleasure in releasing the report.

Don Hopgood

MINISTER FOR ENVIRONMENT AND PLANNING

LETTER OF TRANSMITTAL

TO THE MINISTER FOR ENVIRONMENT AND PLANNING

The Coast Protection Board has much pleasure in submitting to you this report reviewing the metropolitan Adelaide coastal erosion problem and comparing protection strategies.

This report was prepared, at the Board's request, by the Coastal Management Branch. It includes information from studies commissioned by the Board as well as from those undertaken by the Branch.

The Adelaide metropolitan beach is intensively used and loved by South Australians and the adjoining suburbs include some of the most pleasant residential areas in Australia. Not unnaturally, considerable attention is focussed on the Board's work in the preservation and enhancement of the metropolitan coast.

The Report should, we believe, form the basis for the Government's program in this area in the coming years, and as such, it should be available for close scrutiny and Comment by all those with an interest in the coast, particularly the seaside councils. We therefore recommend its public release.

The Board places on record its thanks to staff of the Branch, for the commendable effort in producing this detailed and comprehensive report.

The Board has considered the contents of the report and endorses its recommendations.

Chairman

COAST PROTECTION BOARD

CONTENTS

LIST OF TABLES

LIST OF PHOTOGRAPHS

LIST OF FIGURES

ACKNOWLEDGMENTS

The Branch wishes to acknowledge the help it has received from many people who either provided information, assisted with editing, or helped produce this report. It especially wishes to thank personnel from the organisations listed below and also those retired public servants and others who voluntarily contributed their time to attend meetings and to review material.

- The Adelaide seaside councils and their representatives on the Metropolitan Coast Protection District Consultative Committee.
- The University of Adelaide – Department of Civil Engineering, Department of Geology and Mineralogy, Centre of Environmental Studies
- The Flinders University of South Australia – Department of Earth Sciences
- The Engineering and Water Supply Department
- The Department of Fisheries
- The Highways Department
- The Department of Lands
- The Department of Marine and Harbours
- The Department of Mines and Energy
- The Department of Recreation and Sport
- The Department of Tourism Treasury Department
- The Electricity Trust of South Australia

The Branch would also like to thank the members of the Coast Protection Board for their assistance and support; and the Administration and Technical Services Divisions of the Department of Environment and Planning for word processing, drafting, and report production.

Consultant studies forming a part of this review were carried out by Kinhill Stearns and Riedel & Byrne Pty Ltd, and by Lange, Dames and Campbell Australia Pty Ltd, with Pickands, Mather and Co.

The aerial photographs produced in this report were provided by the Department of Lands.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

THE REVIEW

The intention of this review is to assess and compare all possible alternatives for protecting the Adelaide coast, including the present strategy, which relies mainly on annual beach replenishment. An examination is made of the feasibility, costs and environmental effects of all possible alternatives as well as a review of known information about the processes affecting the coast.

The review extends and updates the Culver Report of 1970. The 1966–70 University of Adelaide study, done under Mr R. Culver's supervision, led to the establishment of the Coast Protection Board and has provided the basis for protection of the Adelaide coast since then. The present review draws heavily on this previous study. This report is intended to provide theoretical support for the Board's work on the metropolitan coast over the forthcoming years, but will similarly require review as more information is obtained on critical factors.

The study was carried out for the Coast Protection Board by the Coastal Management Branch with the assistance of engineering and environmental consultants. A study into the practicality and costs of replenishing beaches with sand from offshore was done by Lange, Dames and Campbell with Pickands, Mather and Co. (the Dredging Study). A consortium of Kinhill Stearns with Riedel & Byrne did the feasibility design and comparison of alternative strategies (the Alternatives Study). Much of the costing of the strategies in this has been revised by the Branch to take account of information that has since become available, and also to take account of some altered assumptions. This has led to different recommendations.

The Civil Engineering Department of the University of Adelaide has contributed through wave studies, which it has carried out for the Board and through comment provided by Coast Protection Board member Mr Culver.

The review is not entirely conclusive because it was deliberately carried out ahead of a survey to find an offshore sand source for beach replenishment. This was done to avoid the possible unnecessary cost of such a survey. A survey has since been initiated, the review having shown that replenishment using an offshore source would be viable and not out of the question economically.

THE EROSION PROBLEM

The present problems are largely the result of building being permitted on slowly eroding sand dunes and have been hastened by this. If no action were taken, beaches and unprotected dunes would be lost, seawalls would be undermined, and public and private property would be damaged. Property losses have been valued at $28m for a 'no action' strategy. The present value of these future losses was estimated to be approximately $10m (excluding the seawalls). The earliest damage would be to some surf rescue and sailing clubhouses, to the Marineland seawater supply, to the Henley swimming pool, and to the seawalls.

Most of the southern beaches, on which 80% of present beach use occurs, would be lost within the 50-year period. Consultants have estimated the present day value of this beach loss to be approximately $23m for the 50-year period.

PRESENT COAST PROTECTION MEASURES

Most of the more urgent seawall construction was carried out in the early and mid 1950s, though some new seawalls and reconstruction of older ones continues to be needed. More recently, the main emphasis has been on the annual beach replenishment program, with sand being obtained mainly from the northern beaches and from accumulations at Glenelg and the Torrens Outlet and being placed on the Brighton and North Glenelg beaches. The average annual quantity moved over the past 6 years has been 105,000 m^3 unbulked (approximately 135,000 m^3 of loose sand in trucks). Stabilising dunes with fences and planting has been a small but important part of the program.

The program has been effective, as shown by the small amount of damage incurred during the severe 1981 storms, but there is still not enough sand to provide attractive and effective beaches at all places along the metropolitan coast. The southern Brighton beach and that at North Glenelg has already been substantially improved, but the north Brighton and South Glenelg beaches are still very low, and there is not enough sand to prevent local erosion problems arising from the sea's interaction with the seawalls.

The sand carting program is not popular with coastal residents, and would be difficult to increase.

THE COASTAL PROCESSES

The main factors having a bearing on the strategies are those that influence the amount of sand on the beaches and its movement. This study has confirmed that the most critical factors are the lack of sand entering from the south, the net northward alongshore sand movement, and the high variability of this movement in quantity and direction and also seasonally and from year to year. It has also shown that both sea level change and loss of nearshore seagrasses could be important factors, though both remain uncertain. The following are summarised conclusions on these and other factors affecting the processes.

Geological Setting

The present beach and dune sands are thought to have been deposited during the early Holocene period (12,000 to 6,500 years ago) when sea level rose rapidly, moving sediments landward from the previously exposed gulf floor. Sea level has been relatively static since then, and there seems to have been very little, if any, addition to the beach and dune sands. The main recent trend has been a northward movement of sand from the original dune deposits at Brighton (now under roads and houses) to form the Le Fevre Peninsula.

Three fault zones cross the coast, and movement across these, may contribute to the change in sea level relative to the land.

Sea Level Change

There has been a slow but steady rise in sea level at Port Adelaide and Outer Harbour. There remains some confusion about tide gauge datum shifts and corrections early this century, but the evidence indicates that the average change over the past 40 years has been just over 2 mm a year, and that this rate or a slightly lesser one has applied over the past 100 years. This rise may be partly due to the general worldwide trend in sea level change and partly due to local land settlement, possibly caused by extraction of ground water, consolidation of clays, movement across the faults, or a combination of these. This settlement is being measured by precise levelling, but will not be known accurately until 5 years time, when re-levelling is done.

An average relative rise of 2 mm/year has been assumed for the review. The most important effect of this is that approximately 5,000 m^3 of sand per year would be needed on the southern metropolitan beaches to counter its effect. The effect of this sea level rise on storm tides and wave heights is small over the 50-year study period, and does not influence the design of coast

protection strategies. Sea level rise is nevertheless the most important factor in the longer term, especially for the lower-lying, northern parts of the Adelaide coast, where flooding is likely to become more frequent, though not significantly so for the next 100 years or so.

Higher rates of sea level rise of up to 7 to 10 mm/year have been recorded, with an average of 7.5 mm/year possibly having applied for a 10-year period. These would have a significant, though hidden, effect on the beach replenishment program, the beach changes being masked by larger seasonal changes.

Possible higher rates of rise associated with the postulated 'greenhouse' warming of the earth have not been taken into account. Larger rises could occur, but are too uncertain to take into account at this stage. The effect of an average annual rate of 10 mm/year rise over the 50-year comparison period has been considered briefly.

Storm Surge

Extreme tides usually occur during major coastal storms at Adelaide, especially when these are preceded by winds from the north-west. The greater inshore water depths allow larger waves at the coast and enable these to reach higher, causing more damage. Both the record tides in June and July 1981 occurred on spring tides and both were approximately 1.4 m above the predicted tides. Although a 1.6 m surge has been recorded at a lower tide, the probability of higher surges coinciding with spring high tides is not great. It seems unlikely that a 1 in 100 year extreme level would be more than 0.15 m above the 1981 levels, or that an absolute extreme would be more than 0.3 m above these levels. Such tides would cause flooding of low-lying areas in the northern part of the study area, but do not significantly affect the coast protection strategies considered here.

The surge is mainly due to the effect of wind on the sea surface, possibly amplified by the gulf configuration and by natural oscillation of the water mass in the gulf. Hydrostatic effects may also be significant, depending on the location of the weather system and on the pressure drop. Other effects, such as very long period ocean waves, may also contribute.

Winds, Waves and Alongshore Sand Movement

Ocean swell waves are small because of their attenuation through the gulf entrance and across the shallow gulf waters, and their effect on alongshore sand transport is slight. Sand is mainly moved by wind waves generated in the gulf. The gulf configuration favours wave generation by winds from the south-west, which is also the dominant wind direction. This results in a net northward sand movement along the Adelaide beaches. However, winds from the north-west quarter are significant, and in some years have nearly equalled those from the south-west quarter, causing sand transport in a southerly direction. The net annual transport is nevertheless always northward. A feature that is very important to the design of coast protection strategies is the high variability in wave energy and direction, seasonally and from year to year. This makes accurate prediction of the effects of structures that trap sediment or modify the coastal alignment impossible, and makes these methods uncertain.

The annual average northward sand transport is estimated at approximately 30,000 m^3/year. This estimate is based mainly on accumulation at structures and on the Branch's experience with the sand carting program. The figure is slightly higher than the estimates in the 1970 Culver Report. Both are within the range of values obtainable by calculation based on wave energy.

The interaction of waves with the seawalls affects the sediment transport rate, as does the temporary beach instability after beach replenishment. Both are relevant to protection strategy.

Waves have been recorded off Seacliff since January 1981, and the data has been used by the University of Adelaide to develop two accurate wave prediction models, enabling waves to be calculated from wind records or predictions. A local sediment transport model was partly developed as an 'extra' to the research project, but not to a stage where it could be used for this review.

Nearshore Seagrass Change

There have been large losses of nearshore seagrasses, mainly between South Glenelg and Point Malcolm, and to a lesser extent in the area between Taperoo and Outer Harbour. The Branch has estimated a loss of approximately 1,400 hectares, most of which has occurred since 1972.

The initial cause of the recession remains uncertain, but is almost certainly related to human influence. Excess epiphytic growth, stimulated by nutrients from the Glenelg Sewage Treatment Works, and possibly also from stormwater, may be a cause. So could sand movements caused by coastal structures. These and other possible causes are discussed fully in the report. Whatever the initial cause, the present recession seems to be mainly due to plants being smothered by mobile sand released from adjacent areas.

This seagrass recession could be having a large effect on the coast by releasing sand for movement either onshore or offshore, and by allowing slightly higher wave energy to reach the coast. It has not yet been possible to measure these effects or to gain an adequate understanding of the processes. Any effect could be obscured by changes due to other causes. There is no firm evidence of either a sediment gain or loss associated with the seagrass recession, and it has therefore not been taken into account in this review.

The Sediment Budget

There is insufficient information available to draw up a complete sediment budget for the study area, though an attempt led to the following useful conclusions:

- The only significant supply of sand into the area is a possible small alongshore drift from the south. This is probably balanced by losses to offshore at the Patawalonga groyne and elsewhere. Erosion of the dunes at West Beach and Minda provides a small supply within the study area.
- Onshore and offshore sand movements are probably small, despite some offshore losses during storms. Their effect is much less than that of alongshore transport and sea level change.
- The northward alongshore sand transport (estimated 30,000 m^3/year) varies along the coast, causing loss of sand from South Brighton, accumulation at North Haven and Outer Harbour, and varying effects between.
- Sand accumulates at the Patawalonga groyne and at the Torrens Outlet, though sand bypassing also occurs at both. The accumulations would represent a loss from the system, were they not used for beach replenishment.
- The assumed 2 mm relative sea level rise is equivalent to a loss of approximately 5,000 m^3/year of sand from the eroding southern part of the study area and to approximately 17,000 m^3/year from the whole study area. The effect of 10 mm rise would be at least 5 times as much, probably more.
- Seagrass recession between Glenelg and Grange may be influencing the sediment budget in quite complex ways, by releasing sand and also by allowing more wave energy to reach the coast. Large quantities of sand may be involved.

The beach replenishment program would, over the past 10 years, have only just offset the alongshore and sea level rise losses, by an average amount of less than 10,000 m^3/year for the assumed 2 mm/year sea level rise. Increased replenishment over the past 5 years would have given a more definite gain over this shorter period.

SAND SOURCES

Offshore

An extensive area of the sea bottom off the southern part of the metropolitan coast was surveyed in 1972 in a search for sand conveniently located for beach replenishment. Sand of similar median

grain size to the beach sand was found, but it contained too much fine material and was mostly under a hard cemented layer. Subsequent, limited, core sampling off Brighton has confirmed this result.

In the absence of other evident deposits, the Branch surveyed a large sand bank north of the Outer Harbour channel and proved approximately a million m^3 of marginally suitable material. However, much of the material is too fine, and losses from the beaches would add considerably to the cost of using this source. Use of an area off North Haven would be advantageous in reducing dredging for both the North Haven entrance and the Outer Harbour channel, but samples from the recent North Haven dredging indicate that this sand is much too fine.

The chance of finding suitable sand offshore is not high, because the material offshore has been shown to be generally finer than that on the beaches. An area off the Onkaparinga is an exception, and will be the main focus of a recently initiated new survey for offshore sand. For costing purposes this review initially assumed three hypothetical sources – Outer Harbour, North Haven, and off the Onkaparinga River. The North Haven source was subsequently excluded.

Onshore

The present replenishment sand supplies on the northern beaches and at the Torrens Outlet are less than had previously been assumed. This is mainly because sand north of the Largs Bay jetty has been found to contain too much fine material. At the present rates of use, the suitable deposits might suffice for only another 10 to 15 years. However, they could last for longer, depending on whether the beach sand was replaced from the shallow nearshore zone, and on how quickly this happened.

Dunes on the Electricity Trust's property at Torrens Island have recently been found to contain enough suitable sand for approximately 10 years of beach replenishment, though the area would need to be carefully rehabilitated, and this would add to the cost. Action is underway to ensure that this sand is reserved for the beaches.

There are no other local deposits of significance other than in dunes at West Beach, West Lakes and Port Noarlunga, all of which are important recreation or conservation areas, and in small accumulations against the structures at Port Stanvac and Glenelg. Use of some of the dunal deposits could become necessary in the long term.

Other possible supplies would cost much more, but are nevertheless continuing to be investigated. These include washed sands from the commercial sand pits south of Adelaide, and distant deposits of coarser sand at Lake Alexandrina and Mount Compass.

CONTROLS ON DEVELOPMENT, TAKING OF SAND, AND ACCESS

The report includes a brief review of controls relevant to avoiding inappropriate development at the coast and those that have a bearing in implementing the strategies considered. The major legislation is the Planning Act of 1982. Controls are also available under the Coast Protection Act (1972), but have deliberately not been implemented, to avoid unnecessary overlap.

Existing legislation is seen to be adequate to prevent most future problems without hindering the timely provision of coast protection works. However, some minor inadequacies in the Planning Act are identified. These relate mostly to the exclusion of certain activities from the approval processes, these mainly being road and drainage works, dredging, earthworks, and low retaining walls. There is also seen to be a need to exclude emergency Government coast protection works from the time-consuming notification and approval process. Other emergency Government works are excluded.

While there appears to be sufficient legal authority for the Board's beach replenishment program, clearer authority would be desirable to avoid unnecessary disputes over the taking of beach sand. An amendment to the Coast Protection Act would be required. There is also seen to be a need for

the Board to be able to prevent the taking of beach sand by others. This could be achieved by regulation under the Coast Protection Act.

It is noted that the 'restricted area' provision of the Coast Protection Act may need to be applied more extensively to protect dunes in the study area, and that it could also be used to temporarily restrict access to parts of the beach where this is necessary for public safety during sand carting or other coastal works.

THE PROTECTION STRATEGIES – CONCLUSIONS

(Comparative costs are tabulated and shown graphically in Appendix B.)

Alternative strategies were designed to a preliminary stage, and present day values of the total costs over a 50-year period for each were compared. A range of discount rates (equivalent to interest rates after adjustment for inflation) was used in calculating the present day values. The main comparison was at a 5% discount rate, which was considered to be the most appropriate. The sensitivity of the cost comparison was tested for discount rates in the range 2% to 10%. Cash flow was also compared, because methods with similar present day values can involve quite different expenditures.

The Alternatives consultant study summed present day monetary values for beach gains and losses, those for the hypothetical Patawalonga dredging saving, and those for construction and maintenance to produce a balance sheet of benefits and costs; however, this approach was deliberately not taken here. It was felt that these 'total' comparative figures could be misleading because of the disparate reliability of items included in them, and also that they might distract attention from other critical aspects. The total benefits and costs can readily be obtained by summing the information in tables 8, 9 and 10 in Appendix B. Comparison of total benefits and costs has nevertheless been made where appropriate and these are discussed in the main report and in these conclusions.

Although considered separately in the main text, environmental and social impacts are included here under the summaries for each of the alternative strategies.

No Protection – Consequences

Coast protection costs of the order of those considered in this report are justified by the likely consequences of taking no action. The estimated property and beach amenity losses for this are $12 and $28 million respectively, and most beaches south of West Beach would disappear within the 50-year study period.

Seawalls Only

Constructing seawalls and not doing any further beach replenishment could be the least costly way of protecting property over the comparison period. The present day value of the construction and maintenance costs was estimated at $10.4m for a 5% discount rate. However, this is essentially a short-term solution. Its cost advantage relies on most of the recently constructed seawalls surviving the beach loss that would occur, and this is by no means certain. It would result in slightly more beach loss than taking no action. This option is not considered viable, because of its disadvantages and cost uncertainty.

Both the 'no protection' and 'seawalls only' options would have severe social and environmental impacts through the loss of important beaches. This would markedly reduce the present opportunities for beach recreation and would adversely affect tourism to the state. When the monetary value of beach loss is taken into account, the seawall option has the highest cost of all the strategies. It is the only strategy that does not have a positive benefit when compared with the 'no protection' option.

Continuing the Present Measures

If beaches are to be retained, continuing the present measures (mainly annual beach replenishment with some seawall construction) has the lowest cost in present value terms. It will be the only viable strategy if an offshore sand deposit suitable for beach replenishment cannot be found. It would also eventually provide much improved beaches.

Most of the seawall construction, mainly in the Henley area, would be completed in the first 10 years. Beach replenishment would be mainly to Brighton and North Glenelg as at present, and would be at the present level of 100,000 m^3 a year. This report also recommends regular moving of sand past the Torrens Outlet.

The eventual aim of the beach replenishment is to establish wider beaches and a strip of dunes in front of the present rock seawalls. In due course, these dunes would contain enough sand to prevent all but the most severe storms reaching the seawalls. In time, this strategy would eliminate those local erosion problems that are caused by the interaction of the sea with the seawalls. It would take approximately 50 years of sand carting at the present level to provide similar beaches and dunes to those assumed for the major replenishment alternatives.

At $13.4 m, its present day cost value at a 5% discount rate is approximately $6m to $8m less than that for a major beach replenishment, which is the next least costly method. At a 7.5% discount rate its cost value is approximately half that of the next cheapest alternative. At lower discount rates, which are less likely to apply, cost value differences between the alternatives reduce. The present day values for continuing the present measures, a major beach replenishment, or a major replenishment with groynes become approximately equal at rates of 2% to 3%.

The present strategy is more flexible than the others, and can be adjusted to take account of beach conditions. While the strategy should not be relaxed during calm years (bearing in mind the intention to build up wider beaches rather than to preserve the status quo), sand quantities during these years could be reduced to offset larger quantities that may be needed during stormy years.

Although it has little impact on the physical or biological environment, continuing the present measures would involve nearly three times as much truck travel as the other alternatives considered, and would have by far the greatest impact on coastal residents. Trucks and loaders would continue to operate on beaches for parts of each year. Few opportunities exist for reducing the impacts. Winter replenishment has some advantages and may be more acceptable to beach users and coastal residents, but does pose some practical difficulties. It will be tried in winter 1984.

A moderate level of sand removal from the northern beaches prevents excessive sand build-up and preserves the amenity of these beaches.

The present beach sand sources at Point Malcolm, Semaphore, and the Torrens Outlet are likely to be exhausted in 10 to 15 years time. A deposit on Torrens Island, north of the Power Station, would provide a necessary alternative supply for approximately 10 years, after which more distant and increasingly expensive sand would need to be used. Known sand sources would suffice for the 50-year period, but would need to be reserved for the purpose.

Use of the Torrens Island sand would need to be carefully planned to avoid damage to the important samphire and mangrove environment on the Island. An environmental impact statement or public

environment report may be required, and the use would need to be endorsed in the Supplementary Development Plan to be prepared for the island.

Status Quo Option

A reduced, 'Status Quo', version of the present measures was considered. Replenishment quantities were assumed to be reduced to match losses rather than to eventually establish dunes and wider beaches. This option would not remove the cause of most of the present problems, nor would it allow for any error in the estimates of sand loss. Sand trucking would only be slightly reduced because replenishment would still be needed in the present quantities at Grange and North

Glenelg. The option offers only a small saving over the first 20 years, and was consequently dismissed.

Although this option has the least cost of the alternatives considered, its overall benefit (compared to the taking of no action) is similar to those alternatives that would provide an improved beach, and higher than for the 'Seawalls only' alternative.

A Major Beach Replenishment

A major replenishment of the Brighton and Glenelg beaches with 3 million m^3 of sand dredged from an offshore source would be a viable option if suitable sand were found. Present day values of the cost of such a strategy are estimated at $19.1m to $21.6m (5% discount rate), depending on the source. This alternative would be worth considering despite the extra cost, because it avoids trucking for the next 10 to 20 years, and because trucking after that would be at a much reduced level. It would provide a higher standard of protection and better beaches soonest and, providing that a sand source could be found in an area without seagrasses, would involve less environmental impact and public inconvenience than the present measures. If a suitable offshore source can be found, the overall benefit (compared to taking no action) could be highest for this alternative because of the value of the improved beaches. However, the difference would be small and within the probable limits of accuracy of the costing.

A sand deposit north of the channel at Outer Harbour would be marginally suitable for a major beach replenishment; however, a larger area would need to be tested and confirmed before this source could be considered. The sand at Outer Harbour is variable with much of it being too fine. As a result, an uncertain amount, probably between 20% and 50%, might be lost from the replenished beaches. A 20% loss has been assumed in the costing. In the event of a 50% loss occurring, the present day value of replacing the lost sand by trucking would be approximately $3m.

A trailer-suction dredge provides the most economical dredging and transportation method in the local sea conditions and over the distances involved. However, the water over the Outer Harbour deposit is too shallow for a dredge of this type and a more costly method would need to be used. There are two dredging options for this site. The better and less expensive method is to win the sand using a large cutter-suction dredge, and to pump it to the replenishment beaches via a pipeline, which would be laid under the shipping channel and above ground along the coast. It would not be economical to retain the pipeline for later, top-up replenishment. The other method would be to use the Department of Marine and Harbours' bucket dredge and barges (supplemented with a new barge), but this would cost more and would involve dumping the sand off the beaches and pumping it ashore. This could damage the nearshore seagrasses.

The Department of Marine and Harbours (DMH) has long-term plans for reclaiming land over the Outer Harbour sand deposit. These plans would be affected if this is found to be the only offshore sand source and if a future decision is made to use this sand for a major replenishment. However, use of this source cannot be recommended at this stage.

The most likely location of a better offshore sand source is off the Onkaparinga River or off the coast south of this. Emphasis in the present offshore sand survey is on this area. A trailer-suction hopper dredge would be the most practical and economical equipment for winning this sand. Such a dredge would dredge sand and fill its hopper at the source, and would then sail to a mooring off the Brighton–Glenelg coast, where it would connect to a pipeline to shore (possibly via a barge containing booster pumps) and pump the sand ashore. The more usual method is to dump the sand off the beaches and rehandle it ashore using a cutter-suction dredge. However, this is more costly here because of the wave conditions, and would damage the seagrasses.

Marine damage at the offshore sand source is possible, though would depend mainly on the extent of seagrass beds at the site. There is also a possibility of some nearshore seagrass damage due to a rapid adjustment of the beach profile.

Groynes or Offshore Breakwaters

Groynes or offshore breakwaters could be technically feasible if used with a major beach replenishment, but not on their own. This is mainly because there is insufficient sand moving into the area and the structures would merely redistribute the scarce amount of available sand. Unless combined with replenishment, they would cause more problems than they would solve.

Groynes or breakwaters would reduce the alongshore sand movement and consequently the need for top-up replenishment with its associated costs and trucking nuisance. They would also reduce the cost of maintenance dredging of a channel at the Patawalonga, assuming that such a channel were to be made, and would help avoid the stormwater drainage problems that may result from beach replenishment. The structures would be located so that the narrowest beaches occurred at the outlets.

Their performance is not entirely predictable, and is likely to be less satisfactory at Adelaide than elsewhere, mainly because of the large amount of variation in wave energy direction. Their use with replenishment could result in an overall lower level of protection than could be gained from replenishment alone, and an erosion risk could be introduced at some places where it does not presently exist. They could not be considered as viable options without full-scale experiment.

The present day value (5% discount rate) of using groynes with replenishment is $2.8m more than for replenishment alone. The difference reduces to $1m if maintenance dredging at the Patawalonga is taken into account.

The margin reduces further for low discount rates, and use of groynes could become economical at low rates of 2% to 3%, though such rates are unlikely to apply. Costs are based on the 11 groynes suggested by the consultants for the Alternatives Study. Extra groynes or seawalls, which might be required if the spacings were found to be too great, could add up to $3m to the present day cost value.

Offshore breakwaters would cost more, and their performance is even less certain than groynes, though their effects might not be as harsh.

A hybrid solution, which uses groynes and breakwaters to the best advantage, is also more expensive than using groynes, and the risks would remain.

Use of groynes or breakwaters would alter the appearance of the beach and its suitability for different types of beach recreation. They would also result in some seagrass damage at and in the vicinity of some of the structures, though this is unlikely to spread or be significant in relation to seagrass recession presently occurring. Erosion would occur north of the northern-most structure, causing eventual loss of beaches and foreshore reserves at Semaphore. This would occur in the distant future, and could possibly be remedied by carting or scraping sand from the North Haven accumulation.

A Groyne at The Broadway

A trial replacement of the existing small groyne at The Broadway with a larger groyne was suggested in the Alternatives Study in the context of a major replenishment with groynes. It should not be considered without a major replenishment. Its main purpose would be to provide information for a ‘groyne’ strategy, which has been shown not to be economical and is not recommended here. It could provide more information on the rate of alongshore sand transport, but this is not guaranteed, and the information would not be worth the cost or the risks associated with such a structure. Additional sand trucking would be needed (to compensate for sand trapped by the groyne), unless the structure were built after a major dredged beach replenishment. The groyne could be worthwhile in the latter circumstances if also built in conjunction with dredging a boating channel at the Patawalonga. It would facilitate and reduce maintenance dredging costs, and would provide a considerable length of wider beach and improved protection south of The Broadway.

Other Methods

Other coast protection methods are either unsuitable for the local conditions or insufficiently developed. These include floating breakwaters, artificial seaweed, and shaped offshore dredging (to modify wave refraction). The groyne and breakwater strategies are partly based on the concept of artificial headlands, which cannot be considered as a separate or different method.

Effect of Mean Sea Level Rise on Strategy

The 'greenhouse effect', a supposed global warming due to accumulation of carbon dioxide in the atmosphere, has repeatedly been postulated as being likely to cause increased world sea levels. Recent papers by leading USA scientists have provided detailed projections of quite dramatic sea level change. These have received considerable publicity. The theories have not yet been fully substantiated, however, and no sudden increase in the rate of sea level rise has been measured, despite atmospheric carbon dioxide levels having been elevated for at least the past half century. Neither the theory nor the sea level projections appear to have worldwide acceptance. A strong case has nevertheless been made, and the possibility of future rapid sea level rise should not be ignored.

The effect of an average rate of rise of 10 mm/year on strategies was considered, this rate being in the middle of the range of the recent projections for the next 50 years. If the rise occurred, it would start more slowly and accelerate, and would have its greatest effect in the second half of the 50-year study period.

Continuing the present measures might only remain viable for the next 20 to 30 years, when sand supplies would be exhausted and when a greater length of coastline would need to be replenished.

A major beach replenishment would need to be supported with top-up trucking at approximately the present sand carting level. It could become necessary to repeat the major replenishment in 30 to 40 years time.

The uncertainties of using groynes or offshore breakwaters would increase with a rapidly rising sea level, making these options less attractive. Their benefits would reduce, as more sand was lost offshore than alongshore.

In time it could become necessary to abandon the beach and to rely on seawalls, which would need to be founded lower and constructed to a higher standard than the present ones. The timing and cost of this is not predictable. This option would be premature at this stage, but would need to be reconsidered if or when the higher rates of sea level rise are confirmed.

In the 100 to 200 year long term some property relocation would be unavoidable.

RECOMMENDATIONS

The review has given rise to the following major recommendations:

1. Continuing the present coast protection measures remains the only viable strategy until an offshore sand source is found. It is also the least costly way to improve the beaches and the level of protection. It is therefore recommended that the annual beach replenishment be continued at the present level of approximately 100,000 m^3 of unbulked sand (approximately 135,000 m^3 in trucks). Seawall construction should be limited to replacement of existing seawalls, and should only be done where adequate protection cannot be provided by the beach replenishment program.

   Information that might lead to a review of this strategy is unlikely to become available in less than 2 years, and the strategy should be continued for at least this period.

2. A ‘status quo’ alternative, involving a reduced level of beach replenishment to match the estimated sand losses, would not remove some present erosion problems and might not provide an adequate level of protection. This alternative is therefore not recommended.
3. The search for an offshore sand source should continue, and the strategy should be reconsidered if sufficient suitable sand is found.
4. Use of groynes or other devices to control alongshore sand movement is inappropriate in the Adelaide conditions, and would have damaging effects. Such structures are consequently not recommended. Groynes could be considered if used with a major beach replenishment but would still introduce considerable risk, would offer few advantages, and would result in higher cost.

   Other recommendations of the review are as follows:

5. The large sand deposit on the Electricity Trust’s property at Torrens Island should be reserved for future beach replenishment. Other, more distant, sand sources should be further investigated and suitable sand reserved.
6. The Outer Harbour sand source should not be considered for a major beach replenishment unless no better source can be found and unless further investigations at Outer Harbour show adequate volumes of coarser sand in the area adjacent to that previously surveyed. The use of this deposit would need to be carefully balanced against the Department of Marine and Harbour’s long-term plans for reclaiming the area over it.
7. The Torrens Outlet should be cut through and bypassed regularly to avoid sand build-up and the consequent loss of sand from the Henley and Grange beaches.
8. A single groyne at The Broadway, South Glenelg, longer than the present one, should be reconsidered in the future, if a major replenishment were to be done and if a channel were to be dredged at the Patawalonga.
9. If or when a rapid rate of increase in sea level is confirmed, increased sand carting, major replenishment, or seawalls should be considered, having regard to the rates of change and the results of the present offshore sand survey.
10. This report should be discussed with the councils in the study area to obtain their views on local issues addressed and to obtain their agreement with the findings. The funding and implementation aspects should be considered jointly by the Board and councils, and it may be appropriate for some of the present policies to be altered.
11. The proclaimed Management Plan for the Metropolitan Coast Protection District should be amended to take account of those recommendations accepted by the Board and recommended to the Minister.
12. Further studies and investigations should be done into the main factors influencing coastal processes and future protection strategy. These areas have been identified in this review to be the rates of alongshore transport, nearshore seagrass change, sea level change, and the search for replenishment sand, both offshore and onshore.

    It was not possible for the review to identify in detail all of the studies required, because some of these will depend on results of earlier studies. However, the following are considered to be the most urgently needed.

The Sediment Budget and Alongshore Transport

Continuation of the beach profile program (including special 5-yearly more closely spaced surveys and underwater observation of sand movement in deeper water); further development of the mathematical sediment transport model; determining the seaward limit of significant sediment interchange (by sedimentological and possibly tracer studies); further investigation of sand movements at the Patawalonga groyne and outlet; and monitoring of the main sand source

beaches to determine accumulation and depletion rates and the response of the nearshore profile.

Nearshore Seagrass Change

Close liaison with the 3-yearly seagrass study to be carried out under the supervision of the Department of Fisheries; continuation of mapping of seagrass change; use of the information to design further studies to determine the effect of seagrass changes on the sediment budget; and investigations into and support of trial seagrass planting.

Sea Level Change

An assessment as to whether or not the present tide gauges are sufficiently accurate, and arrangements for their replacement if necessary; establishment of a new gauge in the southern metropolitan area (possibly at Port Stanvac); use of these records to maintain the best possible up-to-date information on rates of change; continuation of the 5-yearly precise levelling survey to determine rates of coastal land settlement; and a study into the broader effects of sea level change, including flooding at Port Adelaide and effects at other parts of the state's coast.

Sand Sources

Extension of the recently commenced offshore sand survey, if necessary, including further sampling of the area off Outer Harbour; in the event of a deposit being found, the carrying out of an environmental study into the effects of its use; further survey to locate onshore sand deposits; and a special study into methods of working and rehabilitating the Torrens Island sand deposit.

CHAPTER 1: INTRODUCTION

A substantial part of the Adelaide coastline is subject to continuing long-term erosion by the sea, mainly due to northward movement of sand out of the Seacliff, Brighton and Glenelg areas, and partly due to a gradual rise in sea level relative to the land. The seafront sand dunes of this eroding coastline have been built on, and roads, houses, and public buildings are consequently subject to erosion by the sea. Severe damage was caused by each of the major storms of 1948, 1953, 1960 and 1972, and to a lesser extent by the storms of June and July 1981.

Expenditure on storm damage repairs and protection works has been considerable. Since 1972 coast protection works in the study area have cost $4.3 million, of which $3.7 million has been paid by the State Government and the remainder by four seaside local councils. In 1981, special Government funding was needed for emergency protection works and for unscheduled new seawall construction arising from the storms of that winter.

In this context, and having regard to the time elapsed since the previous major study in 1970, the Coast Protection Board decided that a review of the situation was needed, with emphasis on the costing and comparison of all possible protection methods. In October 1981, the State Government approved funding towards studies forming part of this review. Most of the $65,000 approved was used for the two main consultant studies.

The intention of this review was to assess and compare the present coast protection strategy, which relies mainly on annual beach replenishment, with all possible alternatives, and to examine the costs, feasibility and environmental effects of the options. Depending on the outcome, the present strategy would either continue or further, more detailed, investigations would be started towards implementing another alternative. In either event, the review would provide a useful basis for public discussion and Government decision-making.

1.1 STUDY PARTICIPANTS

This study was carried out for the South Australian Coast Protection Board by the Coastal Management Branch of the Department of Environment and Planning. It includes information provided by two engineering consultant firms and by the Civil Engineering Department of the University of Adelaide. It also draws on other studies previously commissioned by the Coast Protection Board, and on information provided by other Government departments and university personnel. A brief overview of the state of knowledge about the study area and of the alternatives available was provided by Beare (1981).

The consortium of Kinhill Stearns and Riedel & Byrne Pty Ltd was commissioned to carry out feasibility design and comparative costing of coast protection methods. This included reporting on social and environmental impacts of alternatives. Because of the frequent reference made to this study, it will be referred to as the ‘Alternatives Study’ or the ‘Alternatives Report’. The full title is ‘Adelaide Coast Protection Alternatives Study’. The firm Lange, Dames and Campbell was commissioned with Pickands, Mather and Co. to investigate the practicality and costs of beach replenishment using sand from offshore sources. This is referred to here as the ‘Dredging Study’. The University of Adelaide Civil Engineering Department has been mainly involved through a wave study it has undertaken for the Board and DMH, and through the development of a littoral sand transport model associated with this. Coast Protection Board member, Mr R. Culver of the Civil Engineering Department, has been associated with the study in a review and advisory capacity.

1.2 THE STUDY AREA

The study area extends 28 km from Kingston Park in the south to Taperoo in the north, and forms a part of the proclaimed Metropolitan Coast Protection District. It extends landward to the extent

that property might be affected by erosion, or that sand and rock supplies, and traffic associated with the transport of these may have social or environmental impacts. It extends seaward to include consideration of offshore sand deposits and the interaction of coast protection strategies with the nearshore seabed and the seagrasses thereon. The study area is shown in Figure 1.

1.3 THE COAST PROTECTION ACT

The Coast Protection Act was assented to in 1972 and the Coast Protection Board was established in the same year. The establishment of a special State Government coastal authority was among the recommendations of a University of Adelaide study commissioned jointly by the then Seaside Councils Committee and the State Government of the time. This study, generally known as the 'Culver Report', was carried out between 1966 and 1970. The frequent reference throughout the present report is either to this short title, or as 'Culver 1970'.

Staffing for the Board was provided through the Coast Protection Division of the then newly established Department of Environment. The Division has since become a branch of the present Department of Environment and Planning.

Although the Coast Protection Act provided the Board with a wide range of duties and a responsibility for the entire state's coastline, protection of the Adelaide coastline has remained the most pressing problem, and the larger part of the Board's funding and research effort has necessarily been directed towards this. In other parts of the state, and to a certain extent in the Metropolitan Coast Protection District, the Board has been able to help councils with erosion control measures to the extent where most problems have been solved and it has been possible to place emphasis on avoiding future ones. Unfortunately, the size and nature of the problem on the metropolitan Adelaide coast demands costly ongoing measures.

The Board is not only involved in coast protection issues, but also in the preparation of management plans for the seven Coast Protection Districts; the improvement of the coast through providing grants and advice to councils; and in all matters relating to preservation and optimum use of the coastal environment. These other aspects of the Board's work are mentioned to place this review in context, and to indicate that the Board's duties are not limited to physical protection of the coastline.

1.4 PRESENT PROTECTION STRATEGIES

In 1972, the coastline had suffered storm damage, and several sections were vulnerable to further damage. Early emphasis was on protection of these critical areas by the construction of seawalls. These sloping seawalls were of 'rip-rap' construction – layered, placed rock with large armour stone to resist wave forces. The seaward toe was placed at a level chosen to allow for beach drop, and a stone filter was provided behind the rock to prevent the landward material washing out and causing slumping. The beach replenishment program was also started at that time with carting of sand from Taperoo to Brighton and North Glenelg in 1973. Since completion of the most urgent seawall construction in the mid to late 1970s, emphasis on the preferred strategy of beach replenishment has been increased. The small amount of damage that occurred in the severe storms of winter 1981 confirmed the effectiveness of the combined strategy, though it has yet to withstand a sequence of stormy seasons.

The Board recognises, however, that the present methods, at their present scale, cannot guarantee absolute long-term protection of all public and private property, and that several trouble spots remain.

FIGURE 1 METROPOLITAN COAST PROTECTION DISTRICT AND STUDY AREA

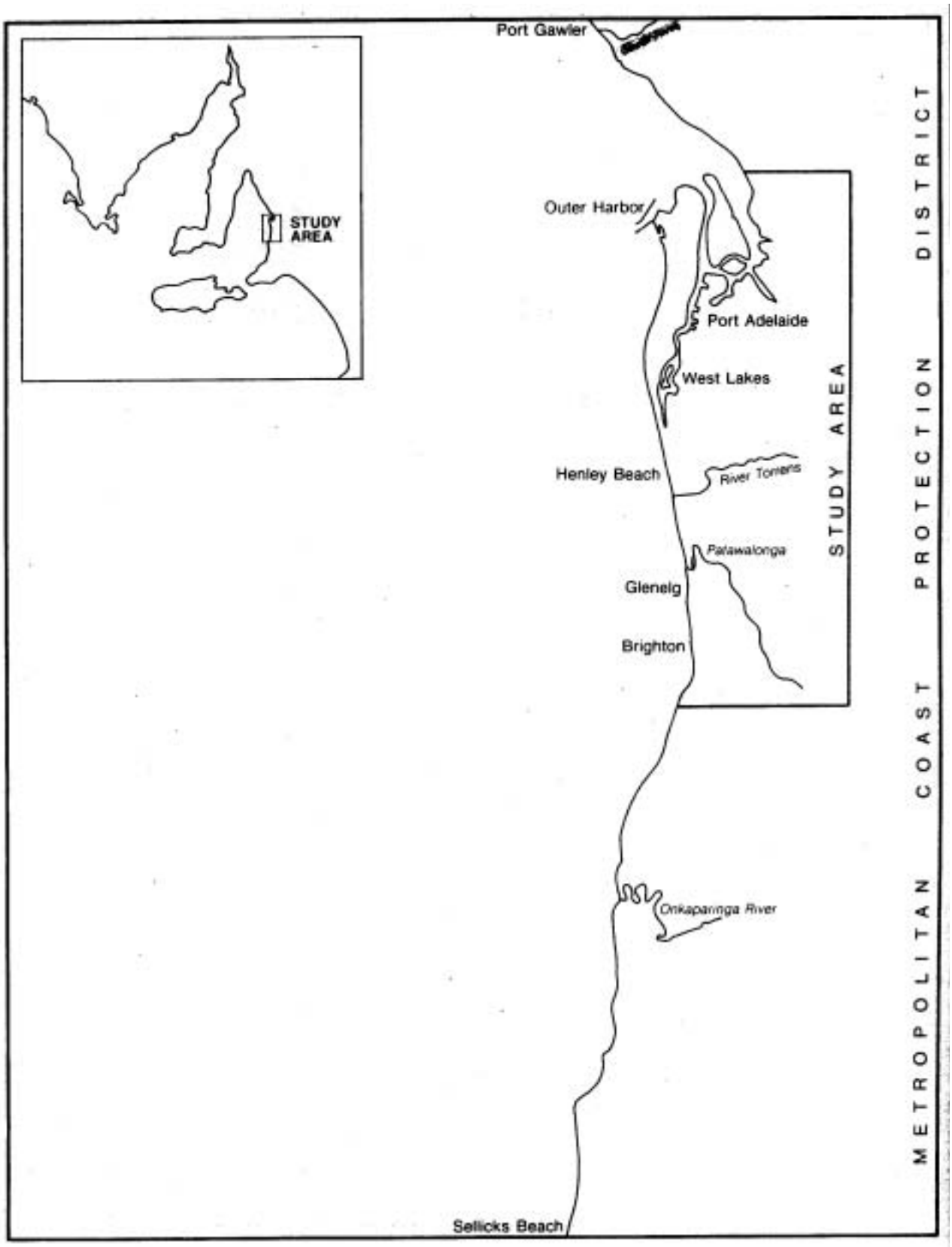
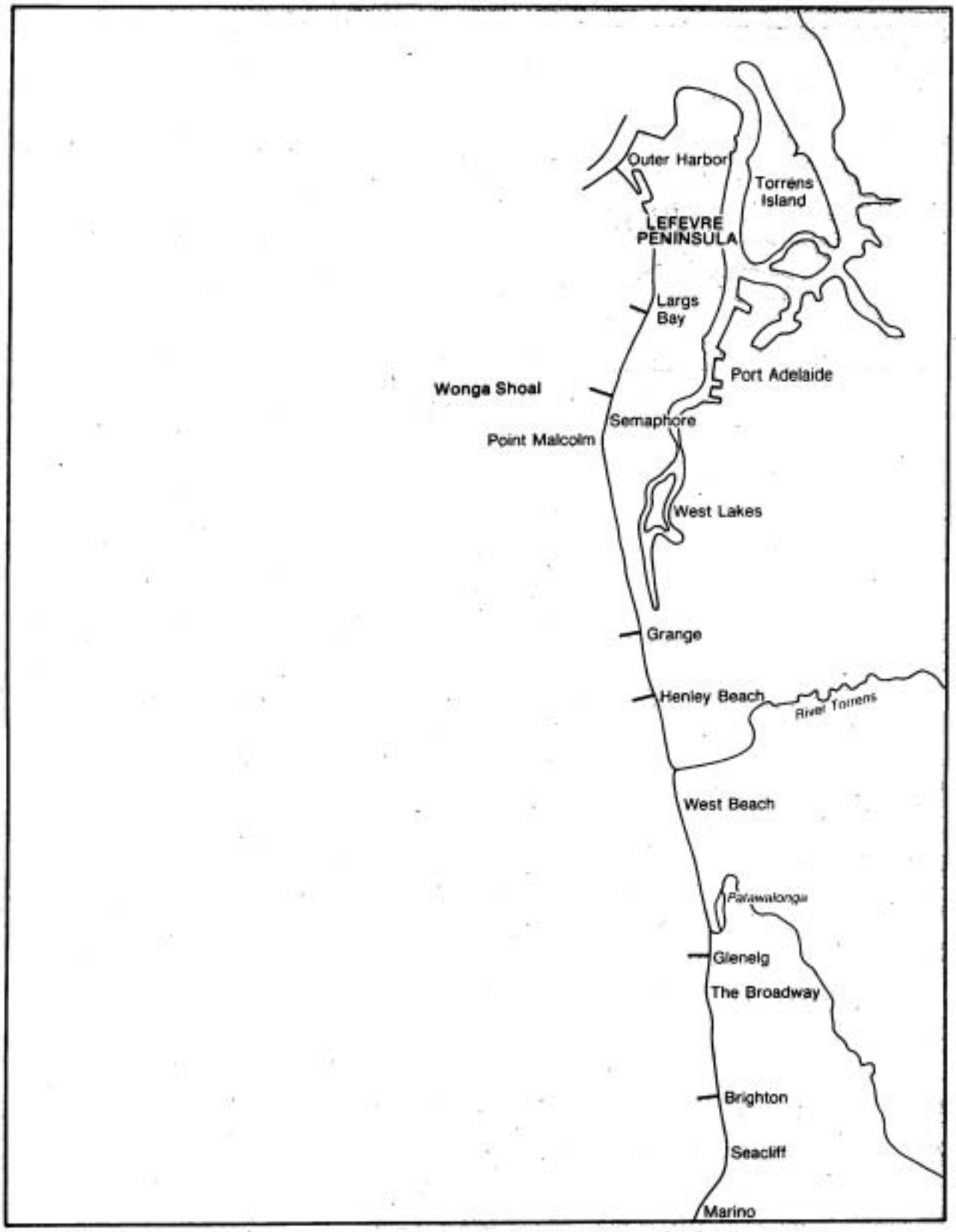
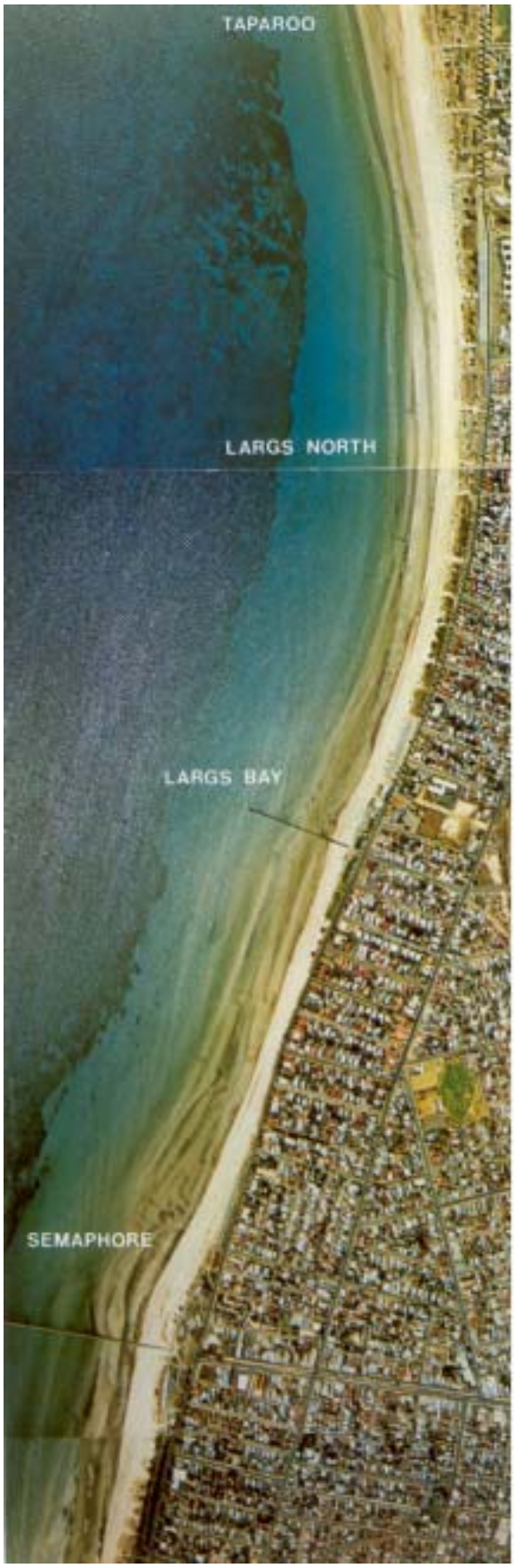


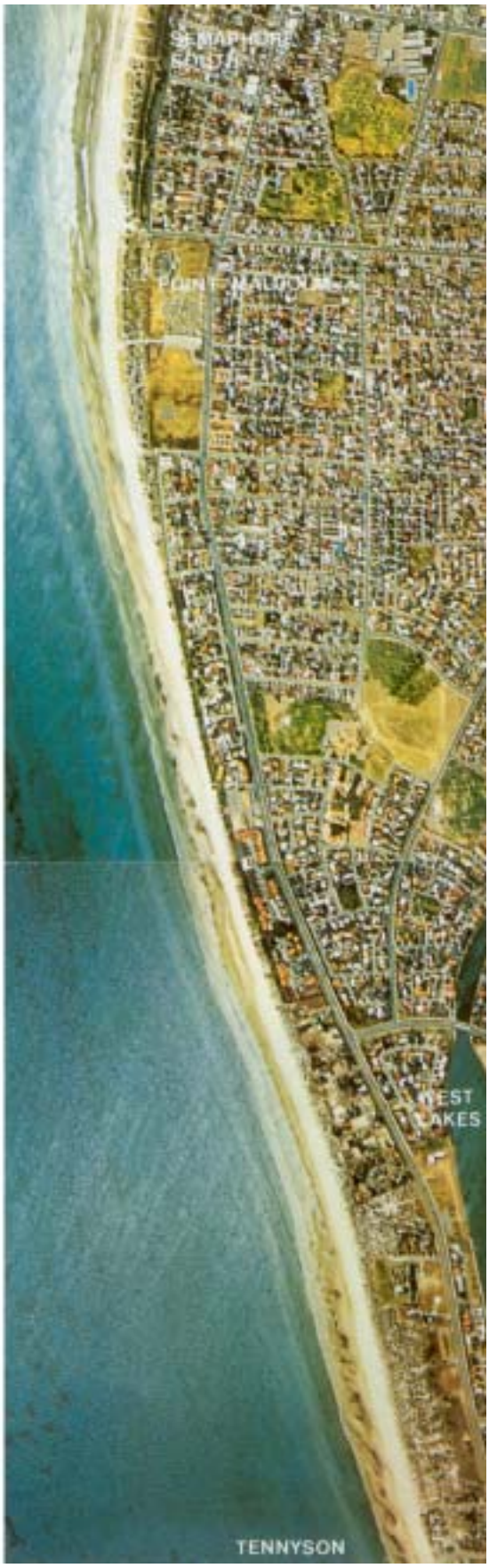
FIGURE 2 STUDY AREA LOCALITIES



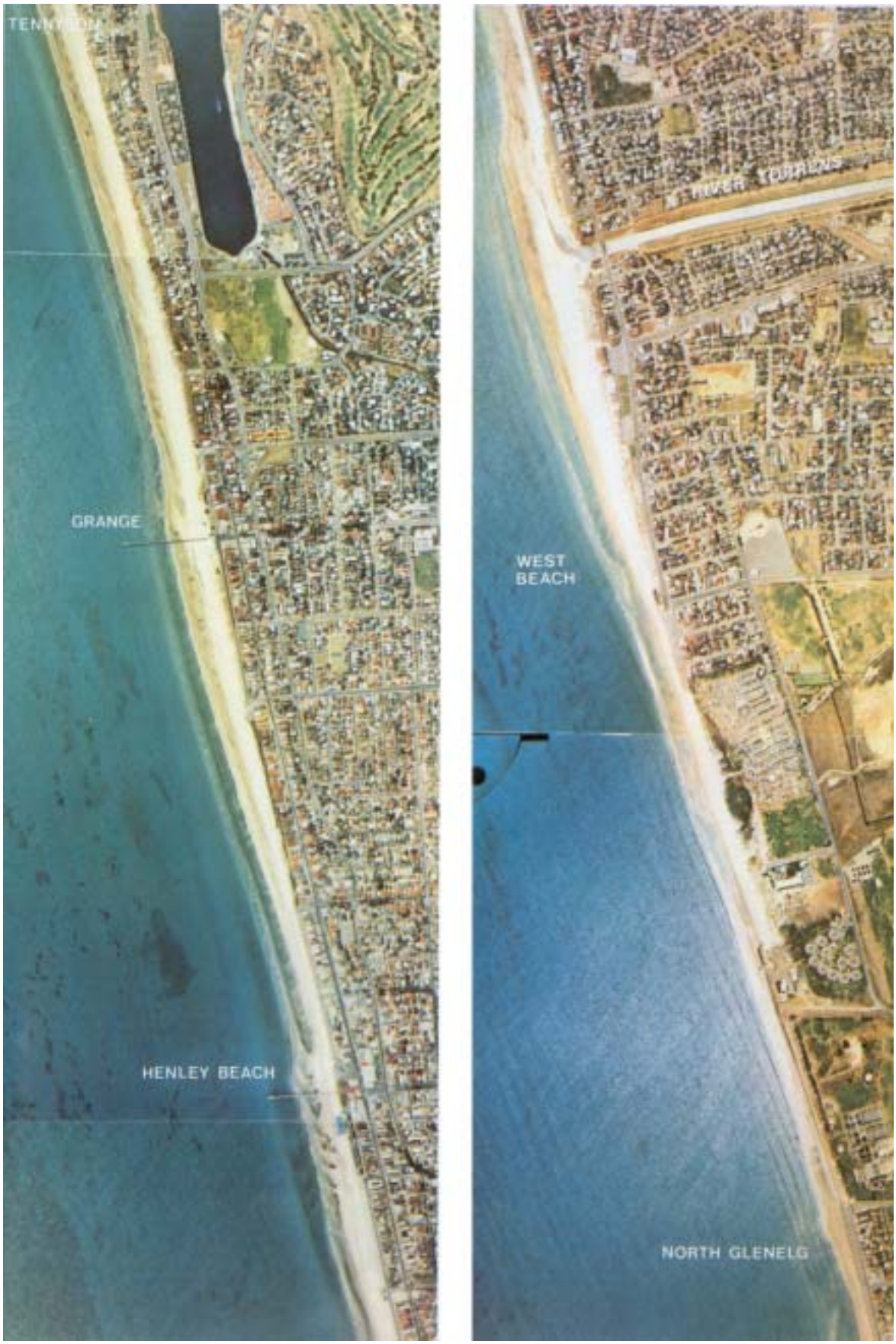
PHOTOGRAPH AERIAL PHOTOGRAPHS OF THE STUDY AREA (A)

PHOTOGRAPH AERIAL PHOTOGRAPHS OF THE STUDY AREA (B AND C)

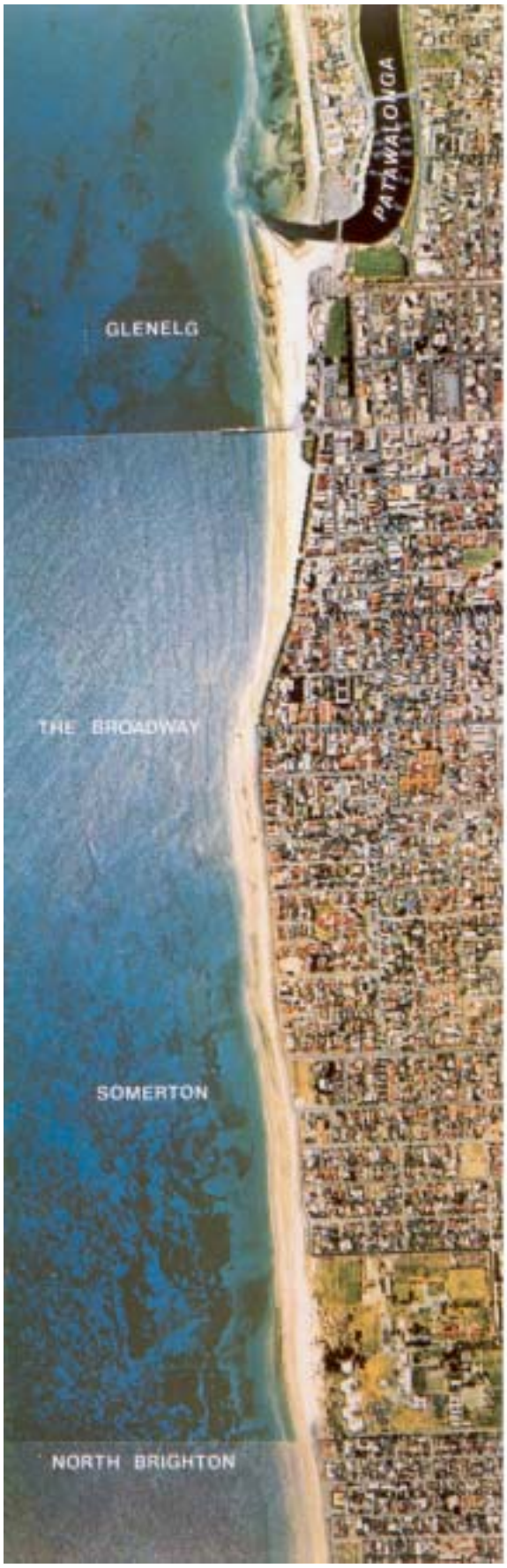


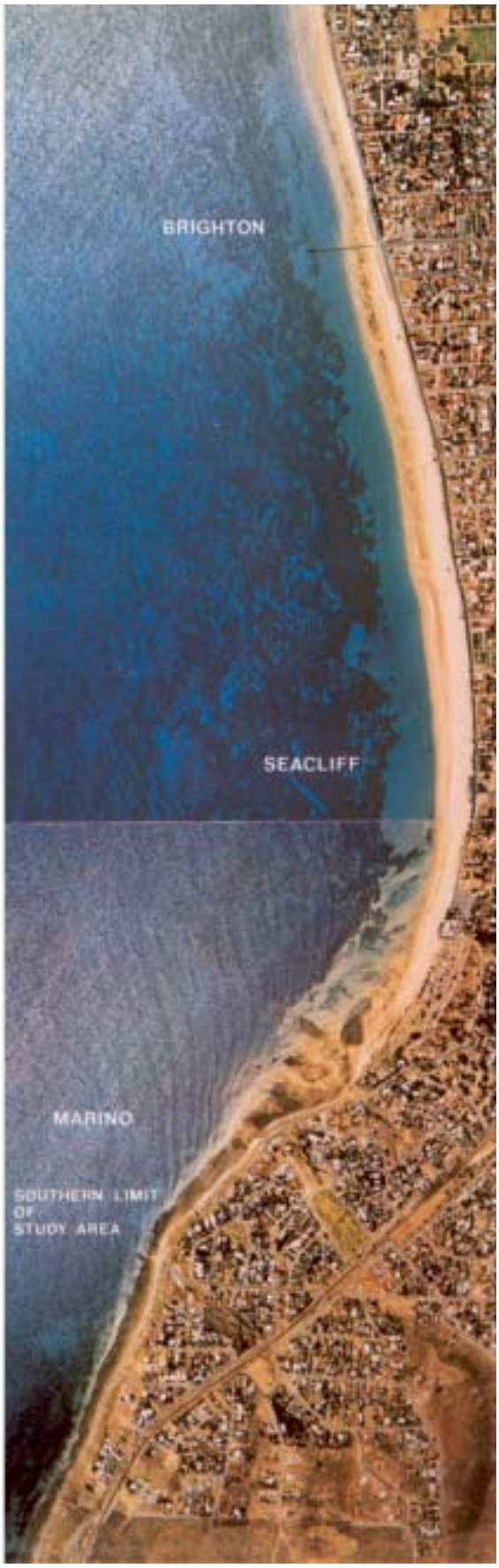


PHOTOGRAPH AERIAL PHOTOGRAPHS OF THE STUDY AREA (D AND E)



PHOTOGRAPH AERIAL PHOTOGRAPHS OF THE STUDY AREA (F AND G)





Further rock protection is still needed at Henley and Brighton to replace sections of older dumped rock, and an increase in beach replenishment is needed if erosion at unprotected parts of the coast such as the West Beach dunes is to be stopped entirely.

The beach replenishment program has been continually questioned by the public, despite public information programs. People find it difficult to believe that the dumping of sand on the southern beaches, to be washed northwards by the sea and replaced by dumping the following year, may be the most economical and effective measure. This has added to the need for the present study.

The current strategy is based on research and costing done in the University of Adelaide study nearly 12 years ago. While the strategy has worked well, it clearly needed to be reviewed to take account of new information about the behaviour of the coast and of changed social and economic conditions. It was also necessary to again explore all possible alternatives to beach replenishment by trucking, as this presents a considerable nuisance to some coastal residents.

1.5 A MAJOR BEACH REPLENISHMENT USING OFFSHORE SAND

The 1970 Culver report's main recommendation for protection of the Adelaide beach was to undertake a major replenishment of the southern metropolitan beaches by dredging sand from offshore. However, this was subject to finding and proving a suitable deposit, and to Government funding.

Several surveys off the Brighton coast have been undertaken, but a suitable deposit has not been found. The search for offshore deposits has continued, but serious consideration of a major replenishment has been deferred because of the high costs and technical uncertainties of using a more distant source. It was therefore important that a reliable assessment of the feasibility, methods and cost of using alternative sources of offshore sand should be a part of this review.

1.6 PREVIOUS STUDIES

This report should be read in proper context to the Culver Report, the Management Plan for the Metropolitan Coast Protection District, and the Study Report on the District prepared by Pak Poy and Associates in 1974. The review extends and updates the Culver Report, though does not supersede it. The Culver Report remains an authoritative reference to the history and understanding of the coastal processes at Adelaide in describing the only major physical study undertaken of this coastline. It forms an important background to the present review.

As required by the Coast Protection Act, the Management Plan sets out 'in general terms the measures that the Board considers necessary or expedient for the protection, restoration or development of the coast comprised in the Coast Protection District'. As such it includes discussion and policies on methods of coast protection, and to a certain extent has pre-judged the findings of this review in recommending the combined strategy of beach replenishment and seawall construction. A draft Management Plan was placed on public exhibition for 2 months in 1978, and the Management Plan was proclaimed an 'Authorised Management Plan' in October 1982. The management plan may be amended to take account of the findings of this review.

The 1974 study report, which, together with the Culver Report, formed the basis of the draft management plan, addressed mainly issues of planning and management, and measures to improve and conserve the coast for public use and enjoyment. It did not consider alternative protection strategies.

1.7 SCOPE OF THE PRESENT REVIEW

The present review, unlike the previous University of Adelaide study, has included very little new field work. It is based on studies carried out by the Branch and commissioned by the Board over the past

10 years, and on review of new developments in the coastal engineering field. Given more time and money, a study of greater depth could have been undertaken and this would have helped to reduce the number of assumptions. However, the review was needed urgently, and it would have been wasteful to collect data relevant to alternatives that might be shown to be out of the question. A decision was made to carry out only the most essential studies at this stage, and to defer other field work until the need was demonstrated by this review.

Studies that have been specially carried out for this review include:

- an assessment of coastline changes from aerial photography;
- the mapping of changes in seagrass occurrence;
- a review of change in mean sea level at Adelaide and elsewhere;
- reconsideration of the sediment budget using information from the Branch's program of surveying beach profiles;
- compilation of historical information on storm damage and on the provision of protective works;
- a re-estimate of littoral drift rates, with special reference to the quantities of sand moved in the beach replenishment program;
- the development of a mathematical model to simulate and predict sand movements;
- a sand tracer experiment in association with the development of this model;
- a trial replenishment using a bottom opening barge and coloured sand;
- the placement of deep benchmarks for a precise survey to measure changes in the level of coastal land;
- the aforementioned two engineering consultancies.

Although some of the strategies considered could result in alteration of the recreational beach environment and have other social impacts, it was decided not to include an opinion survey in the review. It was considered that this would be more usefully done after release of this report.

CHAPTER 2: FACTORS AFFECTING COASTAL CHANGE

2.1 INTRODUCTION

This chapter examines and reports on all the factors that are thought to influence behaviour of the Adelaide coastline. It also provides background information on geology, geomorphology and other physical aspects. The more general, background information is necessarily summarised, and the reader is referred to the referenced articles for more information on these. Matters having direct influence on erosion rates or design of engineering solutions are dealt with in more detail – though even here some summarising has been necessary, and the reader may wish to examine the Branch Technical Reports and other reports commissioned as part of this study. Much of the important background information is contained in the Culver Report. Detail from this is not reproduced here, though some important findings and conclusions are.

The major physical factors that influence the Adelaide coastline are the sand supply to the beaches and its movement along them, and the increase in sea level relative to the land. Major storms also cause large temporary change, and can have longer-term effects by moving sand so far offshore that it cannot be returned by the natural processes. The effect of these factors is readily apparent and easy to understand, though impossible to quantify with accuracy. The effects of some of the other factors, such as seagrass changes, or the effects of seawalls and beach replenishment are less obvious and less well known, but should not be overlooked. This chapter reviews and summarises available information on the local applicability of these factors, as a necessary background to the design of engineering or other solutions.

As will be apparent, the state of knowledge about the behaviour of the coast is limited. Much is still based on theories that have not been fully substantiated, and even where a process such as the littoral sand drift is reasonably well understood, reliable modelling and prediction of quantities has yet to be achieved. Almost all the factors influencing the coastline are subject to climatic or random variation, to the extent that trends and relationships are very difficult to establish. This is demonstrated well by the variation in annual average wind vectors, discussed in Section 2.5.2.

Coastal engineering strategies have too often in the past been designed mainly to take account of the more obvious, short-term, measurable factors such as tides, waves and seasonal beach changes. Inadequate understanding of the slower underlying processes has resulted in these being ignored, with a high failure rate of protection schemes. Rising sea level is the most pervasive of these hidden factors. There is overwhelming evidence that, worldwide, the mean sea level is rising (refer to Section 2.3). This review assumes a 50-year period and, in doing so, undervalues the effect of sea level change. The longer-term implications of strategies will, however, be discussed bearing this in mind.

In considering the planning time period and the effect this has on the importance of the various physical factors, it must be recognised that neither the physical factors nor the economic criteria can be predicted beyond the 50-year period. Prediction is doubtful even within the period. As discussed in Section 2.3, sea level change cannot be reliably predicted in the longer term, and may be subject to unprecedented changes due to the ‘greenhouse’ effect.

Despite these limitations, it is essential that all the natural factors are understood, and taken into account as far as practical in designing and considering options.

2.2 GEOLOGICAL AND GEOMORPHOLOGICAL FACTORS

2.2.1 Geological Setting and History

Adelaide is located on the east shore of Gulf St Vincent, within the St Vincent Basin.

This basin is thought to have been formed in the Palaeocene to Middle-Eocene period, about 55 million years ago, at the same time that Australia separated from Antarctica. Earth movements involving faulting and tilting resulted in the formation of a series of basins including the St Vincent Basin.

About 40 million years ago local tectonic activity increased and reactivated movement along the faults that had been formed during the earlier time of crustal instability. The eastern blocks rose to form the Adelaide Hills and the western ones sank to below sea level (Jones 1981). This situation prevailed until the end of the Tertiary period, about 2 million years ago, and was followed by successive glacial and inter-glacial periods, during which sea level rose and fell dramatically.

During the last interglacial period, approximately 135,000 to 120,000 years ago, the sea was 4 to 5 m above its present level. From 120,000 years ago to 12,000 years ago (the last glacial period) it retreated to levels between 20 and 100 m below the present sea level (Thorn, Bowman & Roy 1981), and is thought to have been at its lowest approximately 18,000 years ago (Jones 1981). At that time, the floor of the gulf was exposed, and a river, to which the Onkaparinga and Torrens Rivers were tributary, flowed across this. Following this fall, a major sea level rise occurred in the early Holocene period (12,000 to 6,500 years ago), submerging the old gulf floor, after which the sea level rise slowed considerably. Waves, in the process of adjusting the gradients of the new sea floor, transported a considerable volume of sediment to the shoreline, resulting in beach and dune development. The beach and dune system would have moved ahead of the rising sea by the various processes of beach adjustment to wave climate, wind blown sand drift, and storm tide overwash. It is also thought that a contributory factor might have been the earlier eastward transport of sand by strong westerly winds blowing across the exposed and eroding floor of the gulf (Jones 1981).

It is likely that the supply of material to the shore diminished as sea level stabilised and as equilibrium gradients of the sea floor were established by waves (King 1972).

The sea is thought to have reached its present level 6,500 to 6,000 years ago, and to have since maintained this level with relatively small, though important, fluctuations. Recent changes in sea level are discussed in Section 2.3. Geological opinion appears to be divided as to whether or not a higher sea level, up to 2 m above the present level, occurred during the mid Holocene period. Belperio et al (1982) argue that the evidence for this is inadequate, and that the sea level history of South Australia is essentially similar to that of the east coast of the continent, where an elevated Holocene level is not thought to have occurred.

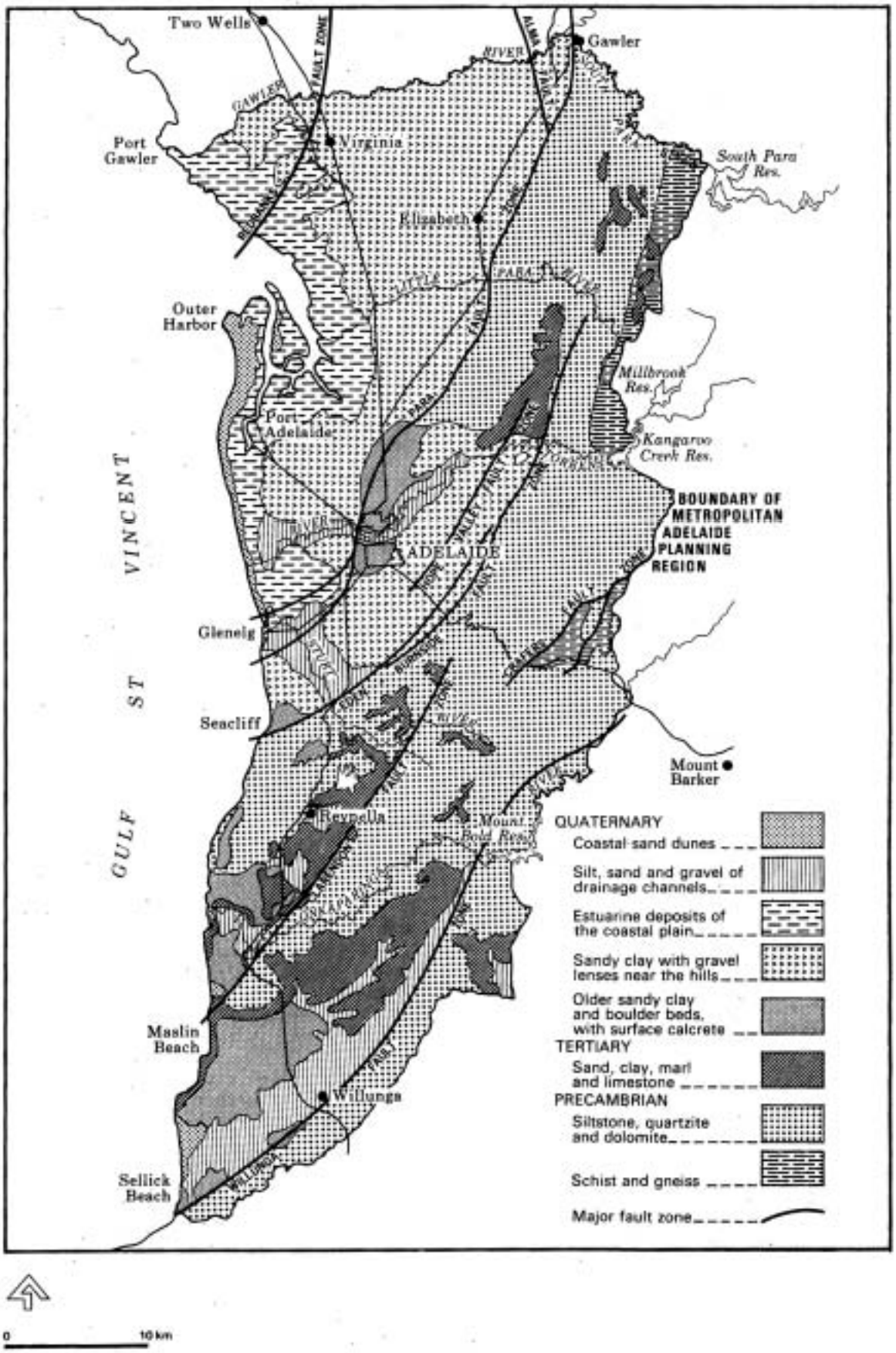
As illustrated in Figure 3, the length of coast in question is intersected by at least three fault zones – the Eden–Burnside, Para and Redbanks fault zones. The possible contribution of earth movement associated with these to settlement of the coast and the apparent rise in sea level is discussed in Section 2.3.4.

2.2.2 The Present Coastline

The study area, between Kingston Park and Outer Harbor, consists of a 29 km length of sandy beach backed by a series of parallel dunes. Kangaroo Island and the shallow Investigator Strait shelter it from ocean waves and swell, and it is subject to medium-energy wave conditions, which together with the medium to fine grain size of the beach sand results in wide gently sloping beaches.

The dunes, which occur as either two or three shore parallel ridges, increase in height from about 4 m at the coast to about 12 m further inland. The width varies between 300 and 500 m. Most of these dunes have been covered with development of buildings and roads.

FIGURE 3 GEOLOGY OF METROPOLITAN ADELAIDE (DEPT OF MINES AND ENERGY)



Although the beach appears to be continuous, it is partially interrupted by several hard horizons, which either outcrop or are located very close to the surface, and by the outlets of the Patawalonga and the Torrens rivers. Artificial structures at Outer Harbor, Glenelg and, to a lesser extent, the various open piled jetties and the small groyne at The Broadway, South Glenelg, influence the shape and behaviour of the coast. Despite appearances, the sand on the beaches and in the nearshore zone is limited to a maximum cover of 2 to 3 m over the underlying clay and calcrete horizons. This cover is much less in a few places and decreases rapidly seawards to as little as a few centimetres or to bare calcrete and clay surfaces.

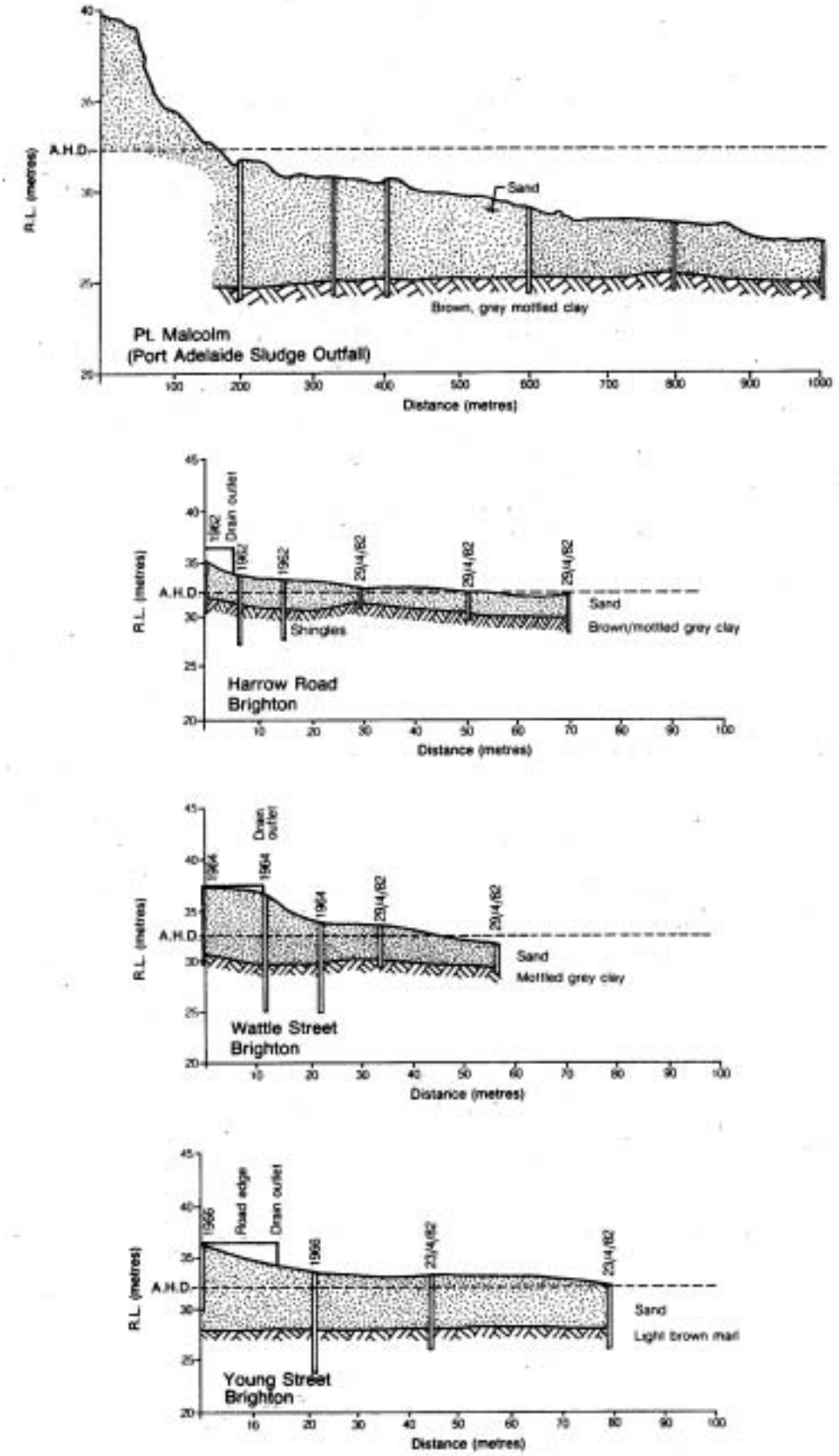
Exposures of the hard horizons occur at Seacliff (in shallow water off Wheatland Street), at The Broadway, South Glenelg, where a rocky shelf (exposed in shallow water) has held the coastline position seaward of the adjacent alignment, and where exposures of mangrove mud can sometimes be seen at times of low beach level, and north of the groyne at Glenelg where a thin layer of stones and sand overlies stiff red clay. Other shelf structures further north at Grange and Port Malcolm are exposed offshore. Although covered with sand on the beach they can be identified by the effects they have on the beach alignment (Culver 1970). Projects undertaken since the Culver Report (laying of the pipeline for the Port Adelaide sewage treatment works sludge outfall, construction of the West Lakes outlet, and the Patawalonga dredging) have provided additional information that has confirmed Culver's observations. All information available on the beach stratigraphy is to be compiled in the Branch and will be issued as a separate technical report. Of necessity, this study has concerned itself mainly with the depths of beach sand in the Brighton and South Glenelg areas, where the information is important to possible protection solutions. Coring was done on the beach adjacent the drains at Young Street, Wattle Street and Harrow Road to supplement the information held by the Highways Department. The cross-sections at these locations are illustrated in Figure 4. These show that the underlying clay beds are almost horizontal, and that a very limited amount of sand exists seaward of the seawalls at these places.

The rocky exposures affect the sand movements both directly by their presence (it is thought that sand may be more rapidly transported across an exposed shelf) and by their influence on the alignment of the coast. The alongshore component of wave energy and hence the sand transport rate varies with the beach alignment. The seaward occurrences of the rocky shelves also influence bottom contours and thus affect wave refraction and the consequent distribution of wave energy along the coastline, though this effect is not significant here.

The accumulation of sand at the northern end of the Adelaide beach system indicates that wind patterns during the mid to late Holocene period (6,000 years ago to a few hundred years ago) were likely to have been similar to those that prevail now. The sand would have been moved northwards by the south-westerly component of the wind and waves until it reached the Port River, where a series of re-curved spits were formed. These progressively built up the Le Fevre Peninsula. The Outer Harbor breakwater, which was built in 1903–05 and has since interrupted this drift, caused build-up of the coast seaward of Lady Gowrie Drive. More recently, the North Haven boat harbour has been excavated out of these sediments. The southern North Haven breakwater built in 1974 has moved the sand accumulation further south.

It should be noted that these northern sand accumulations contain considerable quantities of seaweed. Trial holes dug south of the North Haven breakwater in 1980 showed clear layering, with decomposing seaweed representing about 20% of the volume. The material is washed ashore from the extensive seagrass meadows offshore and its northward drift along the beach is prevented by these part structures. Even taking into account the northward transport, the seagrass deposition in this general area appears to be greater than elsewhere along the metropolitan coast. The factors influencing the shedding of leaves are not understood. There is a correlation with storms, when plants are uprooted by wave action, and there may also be other seasonal effects. The study of seagrass changes described in Section 2.7 identified a dieback of this area, and attributed it to the local progradation of the land – the seagrass retreating as the land advances and the seabed shallows. This may be a partial explanation for the quantities of seaweed on these northern beaches.

FIGURE 4 BEACH STRATIGRAPHY – CROSS-SECTIONS AT PT MALCOLM AND BRIGHTON



The metropolitan coastline is interrupted by two rivers, the Patawalonga and the Torrens. The groyne at the Patawalonga Outlet causes a build-up of sand south of the outlet and a deficit to the north. The artificial Torrens Outlet has caused significant sand accumulation, though it breaks through this periodically and allows part of the sand to move northwards.

The coastline to the south of the study area has some influence on the behaviour of the metropolitan coastline and warrants a brief description. The cliffs that extend from south of the study area to Port Stanvac are of relatively hard material and, although they are eroding slowly, contribute little sediment. The cliffs are fronted by wave cut platforms and relatively deep water. Small pocket beaches, of which Hallett Cove is the largest, occur. Further south the cliffs are softer and are eroding actively. Analysis of beach sands in the area (Culver 1970) show that these sands are coarser than those on the metropolitan beaches and have a lesser calcareous content. Only a small quantity of this sand is considered likely to reach the metropolitan beach. The southern beaches, eg Port Noarlunga and Christies Beach, are aligned at right angles to the predominant wind and wave directions, and therefore tend to hold their sand. Accumulation at the rubble mound structure at Port Stanvac shows only very slow rates of sand accumulation. A further factor that limits movement is the depth of water off the cliffs between Port Stanvac and Hallett Cove.

2.2.3 The Beach and Dune Sands

The University of Adelaide study in the late 1960s included comprehensive sampling and analysis of beach and nearshore sands between Outer Harbor and Sellicks Beach, the results of which are documented in the Culver Report. The main work done since then has been in the offshore zone, and in the search for a beach replenishment source. This later work is described in the next section. The description here of the beach and dune sands is therefore limited to a brief summary of this part of the Culver Report.

Beach sand analysis provides important information towards determining the origin of the sands, and the way in which they are influenced by wind and waves. Knowledge of the sand characteristics is also essential for the design of beach replenishment schemes, where imported sand must be chosen having regard to the grain size and grading of the existing sand.

In the University of Adelaide study, surface samples were obtained at 62 locations between Outer Harbor and Sellicks Beach, these being on the lines of the original beach profiles done by the University. At each location, samples were taken from the High, Mean and Low Water Marks and in shallow water using a pressed in box technique. The samples in deeper water were taken from all jetties using a small grab sampler and from a small boat using the same technique. Sieving analysis was carried out and the samples were leached with acid to determine the calcareous content. Because of the scatter of results, analysis was also carried out on a mixture of all the samples between Marino and Outer Harbor and separately on a mixture of samples between Hallett Cove and Sellicks Beach.

The report concluded from the results that the whole metropolitan coastline consists of ‘a surprisingly uniform sand with a significant calcareous content’. The average median grain size in the metropolitan region was found to be 0.20 mm, ie a fine to medium sand, and there was an indication, not confirmed statistically, that the grain size decreases slightly to the north. Figures from the Culver Report are reproduced here as figures 5 and 6. Figure 5 shows the variation in grain size along the beach, and Figure 6 shows the ‘grading’ or sand size distribution for the combined samples. Detailed information is not provided in the report, but the comment is made that the sand size distribution showed little variation between samples. This suggested that there is very little local wave concentration, which would have the effect of altering the size distribution at the point of concentration.

More recent analysis of source sand for beach replenishment, carried out in October 1982 and January 1983, confirmed that there was very little variation in grain size along the coast between Seacliff and Semaphore, although there was sufficient variation to indicate replenishment overfill ratios of up to 1.35. This figure indicates that up to 35% of the replenishment sand might be lost from the replenishment beach because it is finer than the natural sand on that beach. This recent

sampling did show, however, that the sand at Largs Bay, and especially the sand that has accumulated at the breakwater at North Haven, is very much finer. The North Haven sand has a median grain size of 0.11 mm. At North Haven similar results were obtained from samples in the tidal zone and from samples dredged from approximately 200 m offshore. At the time of sampling, dredging was under way as a part of the maintenance of the entrance into North Haven and samples were taken from the dredge discharge. These results are significant because material from future North Haven entrance dredging has been considered for future beach replenishment use. Offshore dredging alternatives considered in this review have also assumed a borrow area in the general area. Indications are that the material may be unsuitable, because it is too fine.

FIGURE 5 MEDIAN GRAIN SIZE VARIATION (CULVER 1970)

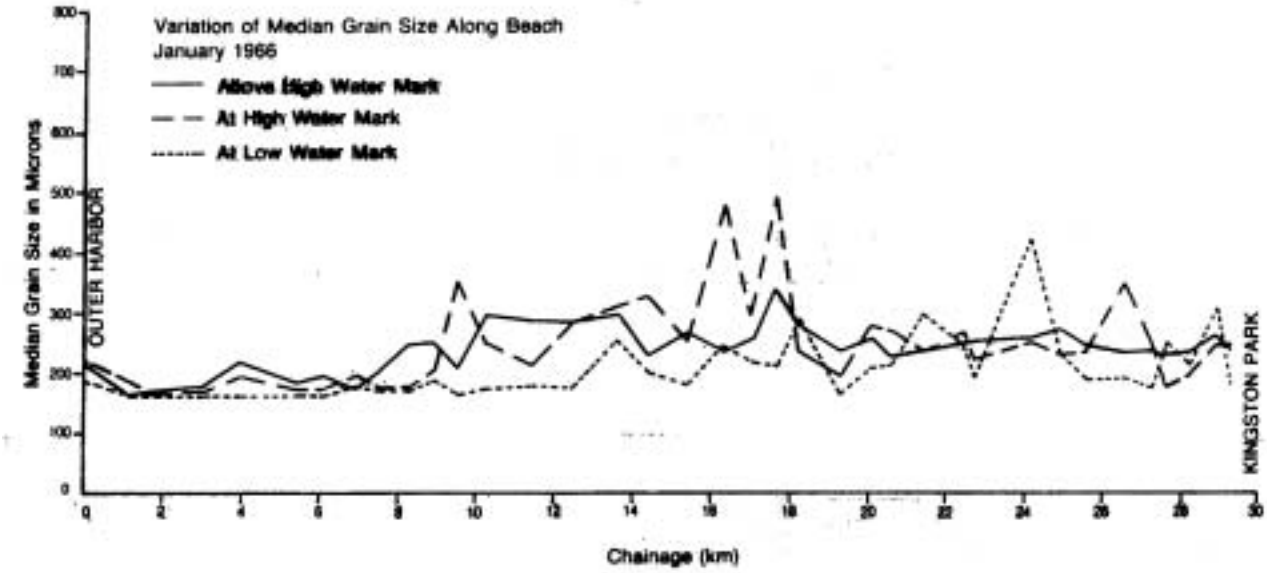
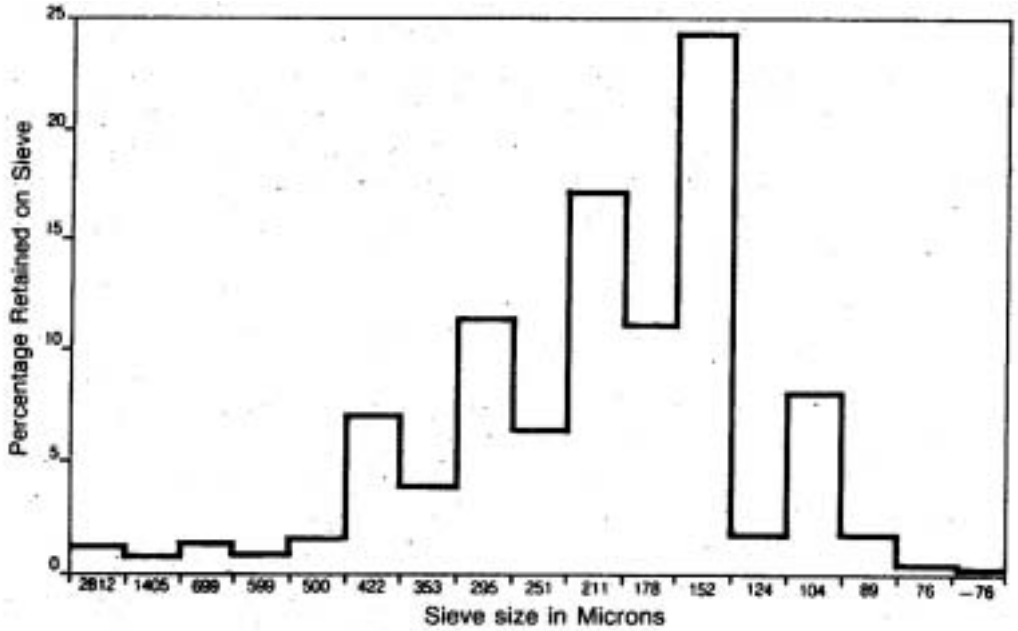


FIGURE 6 SAND SIZE DISTRIBUTION (CULVER 1970)



OUTER HARBOR TO MARINO

Culver's analysis showed the average median grain size for the southern region from Hallett Cove to Sellicks Beach to be 0.35 mm, ie considerably larger than that for the metropolitan beaches. This is attributed to the higher incident wave energy on these southern coasts, and the greater sorting caused by this. A recent Branch analysis of sand accumulation at Port Stanvac, however, has shown grain size there to be very close to that of the study area. This is discussed in more detail in Section 2.6.3 under the sub-heading 'Cliff Erosion and Alongshore Transport'. Culver's offshore sampling, carried out to a depth of 4 fathoms, likewise showed uniform distributions. It showed coarser deposits off Kingston Park, though the distribution of these indicated that the coarser sands were not moving shorewards.

Grain size variation along the Brighton, Henley and Largs jetties was examined by Culver. There was notable sorting in the surf zone at Brighton and Henley, but not at Largs. The sand at Brighton and Henley was found to be coarsest at the top of the beach, finer between approximately 60 and 150 m from the top of the beach, and coarser beyond this.

The calcareous content of the sand was found to be variable, but quite significant for the metropolitan beaches. The average value of 9% considerably exceeds the average of 2% for the southern beaches. This provides a further indication that very little, if any sand from the southern beaches arrives at Brighton. Variability in the calcareous content was thought to be due to some local wave sorting and due to local shellfish beds. Deposition of calcium carbonate from coralline algal epiphytes on the seagrasses could also be a factor (Zimmerman 1973).

The recent testing of replenishment sand sources along the metropolitan coastline showed a similar average calcium carbonate content between Seacliff and Grange, but indicated much higher values in the northern part, between Port Malcolm and North Haven. Between Seacliff and Grange the values varied from 6.2% to 11.5% at mid tide mark, and from 3.3% to 5.0% above high water mark. From Port Malcolm to North Haven the values at the mid tide mark were in the range 11% to 29.2%. Material dredged from south of the North Haven breakwater showed a 31% calcium carbonate content. Both the recent survey and the University work showed a seaward increase in calcium carbonate content. The high offshore values obtained at North Haven are not inconsistent with some of the low water mark values obtained by Culver.

In his report, Culver looked forward to the completion of a more detailed analysis of the sediment data collected during the University study, but this was never completed – though the basic data is still held at the University. Further work on this was not considered a necessary part of this review, having regard to the adequacy of the information in the Culver report and to other priorities. Future analysis, to include skewness and kurtosis comparisons, might provide more information on the origin and movement of the sands.

2.2.4 The Nearshore Zone – Search for a Replenishment Sand Source

Knowledge of the nearshore landforms and sediments can be important to an understanding of coastal processes. Although most of the sand movement occurs along the beach face as a result of wave action, offshore features, currents, sources of sand or depositional 'sinks' can be as important. The 1970 University of Adelaide study was unable to include this aspect in depth, though as discussed in the previous section it did include sand sampling and analysis along all the metropolitan jetties and along survey profile lines to a depth of 4 fathoms (7.3 m).

Notwithstanding the importance of obtaining information on nearshore sediment movement, the main emphasis in offshore work carried out and commissioned by the Board has been to locate a source of sand for replenishing the Brighton beaches. This followed recommendations in the Culver report that investigations into offshore sand deposits be carried out, and that beach replenishment be adopted as the main protection strategy. The report recommended two areas, north and south of the mounds at Outer Harbor, and in approximately 10 fathoms (18 m) of water off Hallett Cove, and indicated that other, more convenient, reserves might exist, but that these were not likely to be extensive. Having regard to the economies of using a borrow area close to the southern beaches, the first survey was of the nearshore area between Henley Beach and Marino Rocks.

This first survey, carried out in early 1972 by Laser Electronics Pty Ltd, pre-dated the Board. It was commissioned by DMH for the Beaches and Foreshores Committee. This Committee, which comprised most of the early Board members, was established by the State Government in 1970 as a temporary advisory body and was disbanded when the Board was established. The seismic, sub-bottom survey included limited sampling and was somewhat inconclusive, though it claimed to established a reserve of 2.47 million m^3 of suitable sand. It was followed up in 1973 by further sampling, carried out by DMH for the Board, and later, in 1977, by limited vibrocoring done by the University of Adelaide Centre of Environmental Studies. This work is described more fully later in this section.

At this stage indications were that, even if a sand reserve did exist off Brighton, it was overlain by clay and/or rocky shelves in places and contained considerable quantities of fine material, which would have to be removed before it could be used on the beaches.

The Coastal Management Branch then carried out a survey of sand deposits in the Outer Harbor area. This work, which was done between 1978 and 1980, established that an adequate quantity of sand was present, but that even the best parts of the deposit were of grain size finer than that of the beaches. Computed over-fill ratios were in the range 1.2 to 1.5, which, together with the long haul, makes this deposit economically unattractive. Other possible reserves, off North Haven, off the Onkaparinga mouth, and off the Port Stanvac/Hallett Cove coast, have not yet been investigated.

Most of the protection alternatives considered in this review involve a large sand replenishment, and the inconclusive situation with regard to the offshore sand deposits was clearly unsatisfactory. However, a reliable investigation of these reserves would have been costly – probably exceeding that of the present review. It was therefore decided that the review should proceed on the basis of several assumed borrow areas, and that the proving of these be deferred until the review had established that replenishment from offshore was a practical and economical method of protecting the coastline.

The review has assumed that there is no suitable deposit off Brighton. This was decided after convening meetings with local experts (Dr Hails and Dr Gostin of the University of Adelaide, and Professor van der Borch of Flinders University), Mr Culver, and retired State Government personnel who had been involved in the early surveys (Mr Sainsbury and Mr O'Malley – respectively, former Director-General and Chief Engineer, DMH – and Mr Buenfeld, former Executive Engineer to the Board). These meetings confirmed the doubtful nature of the Brighton deposit. It was noted that a single sample of clean, ideal sand had been obtained by DMH, but that it had not been possible to relocate the site. From a geological viewpoint, it appears likely that a narrow Pleistocene 'shoestring deposit' exists offshore. The expert opinion was that further seismic work would be worthwhile.

In 1980, the Board commissioned a study of the seafloor morphology in the nearshore zone by a group from Adelaide and Flinders universities, led by Dr J Hails. The analysis of the survey work was delayed and the study report completed during this review. This broader investigation included a study of surface elements, bottom and sub-bottom features. It is described more fully later in this section.

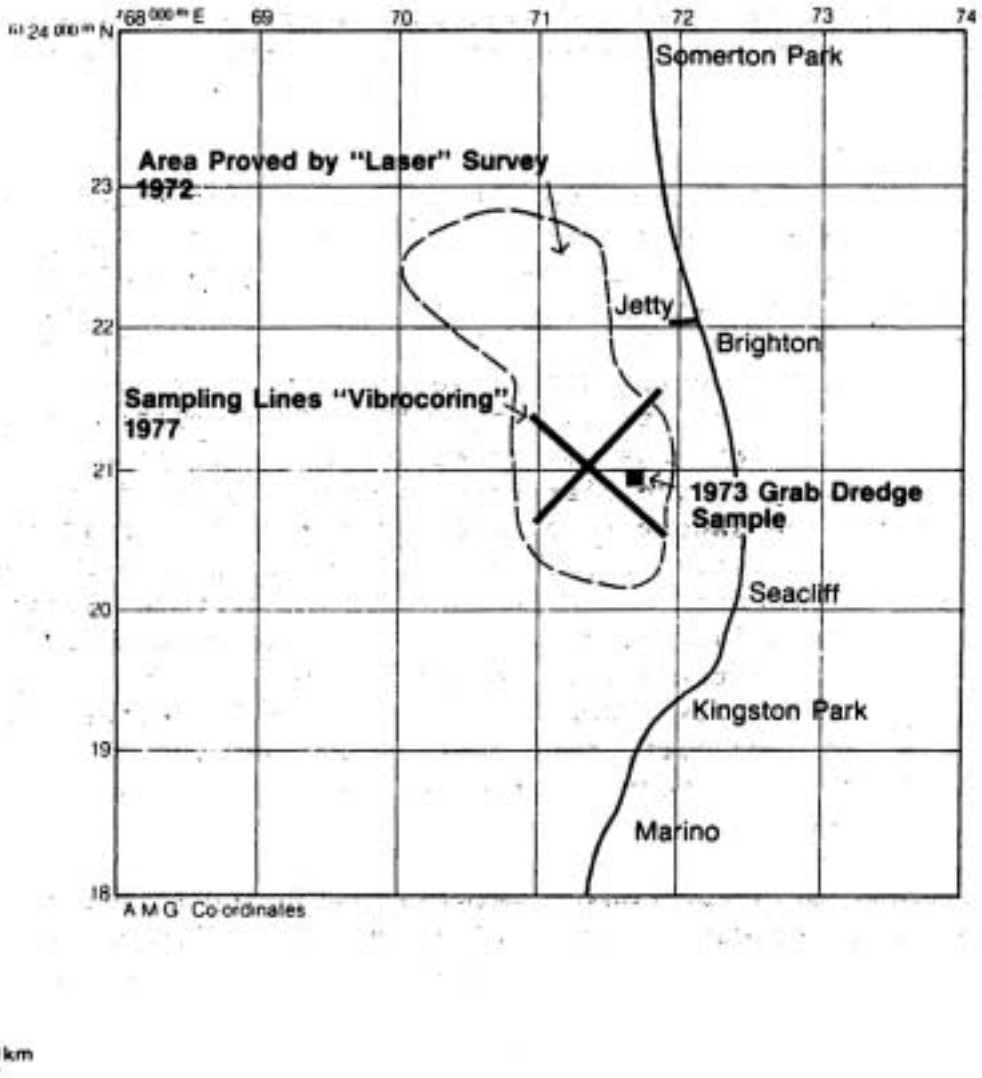
Various studies of a biological or other specialist nature have been carried out by universities and other Government departments, chiefly the Engineering and Water Supply Department and the Department of Fisheries. These have been mainly directed to the condition of underwater flora and fauna and to pollution levels. Relevant studies are discussed in Section 2.7 (Seagrasses). A notable amount of more general offshore research has been carried out by Mr Reg Sprigg, or by companies with which he has been associated, using the vessel MV Saori. This work, which was carried out between 1946 and the early 1970s, included biological and sedimentary mapping of the entire St Vincent Gulf. In late 1979, Mr Sprigg provided the Board with a report on his studies and with some observations based on them. His main observation relevant to he metropolitan coast was that an extensive area of fine to medium grained siliceous sand is located off Port Stanvac and that this might be a source of natural supply to the metropolitan beach. He confirmed findings of the Culver report in respect to the rate of accumulation of sand at Outer Harbor, the lack of supply from the southern cliffs, and the lack of other sources of sand supply.

Sand Source Surveys off Brighton

a) The Laser Survey

The study carried out by Laser Electronics Pty Ltd in 1972 (Brown 1972) covered an area between Henley and Marino Rocks extending along 16 km of coastline and up to 5 km offshore. The area covered is shown in Figure 7.

FIGURE 7 SAND SOURCE SURVEYS OFF BRIGHTON



The survey vessel was equipped with seismic sparker equipment, depth sounder, and a vibratory, air lift type drill. The equipment enabled initial estimates of the thickness of sediment overlying bedrock, and a degree of physical checking on the seismic estimates. The survey was carried out along 167 lines, generally spaced at 400-foot (122 m) intervals along the coast, with selected infill lines at 200-foot spacing. Lines commenced at 6-foot (1.8 m) water depth and extended to 40-foot (12.1 m) depth, the latter being approximately 5 km offshore in places. Several tie lines parallel to the shore were included.

Navigation and position fixing were done by sailing along a narrow shore-based laser beam and using a sextant to measure cut-off angles to shore stations. This was tied in to a shore survey and plotted to National Grid coordinates. Accuracy was claimed to be ±10 feet (3.05 m). The seismic and depth readings were made at 2-minute intervals, this corresponding to distances of between 750 and 1000 feet (230 to 305 m).

Drilling was done separately both for reconnaissance and checking purposes and to prove a 1,000,000 cubic yard sand volume, as required in the specification for the work. It proceeded along the same profile lines used for the seismic work and locations were selected as close as possible to previous seismic recording positions – and generally on a 1000-foot (305 m) grid, with a closer, 200-foot (61 m) spacing being used at selected areas of sand accumulation. The closer spacing was also required on account of the unpredictable occurrence of hard cemented sand (calcrete) or clay layers above the seismically identified 'bedrock'. 192 holes were drilled, and 126 of these were fully 'detailed'. Sampling was at 5-foot (1.52 m) intervals and also where a sediment change was obvious from inspection of the drillings. Sieving analysis only was done on the samples. The results of the survey were presented mainly in the form of isopach maps showing the interpreted thickness of available sand, and it was noted that these were not in absolute agreement with the drilling results – mainly because they do not show the hard horizons encountered in the drilling.

The general findings of the survey were that the present seabed of sand and clay lies over a series of ridges and valleys of approximately 3 m depth, though some of the valleys extend to approximately 6 m depth. Surface sands of thickness '6 inches to several feet' were found to be coarse and pebbly with a high shell content and, in places, a high organic content, presumably derived from seagrasses. It should be noted that the large surface grain size is not confirmed by subsequent survey work, though this latter work has confirmed the observation of bi-modality, particularly in the coarser sediments, as noted in the laser survey. The surface sand was found to be generally underlain by layers of clay, cemented sands or pebbly conglomerates, though these hard horizons were not continuous. Finer grained sands were generally found under these near-surface deposits, though these were also intersected by thin hard horizons. These finer sands had an average mean grain size of 0.20 mm (range 0.15 to 0.30 mm) and contained a considerable proportion of fines, ranging from 3% to 30%. The layers were reported to occur in thicknesses of 1.5 to 3 m and in places up to 6 m.

An area off the South Brighton coast (between the Brighton jetty and Wheatland Street, and between 1.5 and 2.7 km offshore) was considered the most favourable, and was used for calculating the required volume. It was estimated that 2.47 million m^3 of sand could be won from an area of approximately 0.6 square kilometres, in water depths between 8 and 15 m (below low water). Figure 2.6 shows the location, and also where subsequent digging and drilling was done to check the sand source.

This study did not assess the suitability of the sand for beach replenishment, though the possible unsuitability was implicit in its recommendations. These recognised that de-sliming would be needed to remove the fines and that methods of avoiding pollution due to this would need to be devised. It also noted the presence of the hard layers and the possible difficulties of dredging the underlying sands. Subsequent advice from dredging contractors suggests that the hard layers would present no obstacle to a modern cutter-suction dredger. However, the fines content and the avoidance of the upper, surface layers would present problems. The hard layers have proved to be a real impediment to further sampling in the area.

b) DMH Sample Dredging

In 1973, the newly formed Board commissioned the Department of Marine and Harbours to carry out exploratory dredging in the area identified by Laser Electronics. Records of this work are not available, but correspondence and discussion with the personnel involved indicates that the dredging failed to throw more light on the suitability of the sand source. The sampling showed surface sediments as described in the Laser Electronics report, and broken-up limestone. One small sample of clean medium grain size sand was obtained, but the deposit may have been very small, as attempts to relocate on the same position and obtain more of this material were unsuccessful.

c) Vibrocoring

No further work was done on offshore deposits until 1977 when the Board asked the University of Adelaide Centre of Environmental Studies to do experimental coring using their large vibrocoring rig. This work was intended as much to test the equipment for this type of use as to obtain further information about the deposit off Brighton. The vibrocoring method, which is usually limited to soft

sediments, relies on a steel tube being driven into the sediments by a high frequency vibrating weight. The samples are carefully extracted from the tube, care being taken to avoid the sample becoming mixed, compressed or elongated. The vibrocoring rig was mounted on the DMH vessel 'Captain Baddams' for the survey.

Nine holes were attempted on diagonal lines across the area identified by Laser Electronics (see Figure 7), but only two managed to penetrate to a useful depth – presumably at discontinuities in the hard near-surface layer. Cores that were obtained from these confirmed that a fine to medium grain size sand, similar in appearance to the beach sand, occurred under a surface layer of shell and clay. This surface layer was approximately a metre thick. The other attempted holes struck the hard near-surface horizon and were unable to penetrate this. The thickness of sediment above this varied from a few centimetres to a metre.

Although more holes had been intended, the survey was terminated because of unfavourable weather and because of the inability of the equipment to penetrate the hard layers.

The Coastal Management Branch report on the work concluded that:

> The short drilling program undertaken neither confirms nor disproves the assertion by Laser Electronics that a body of sand sufficient and suitable for beach replenishment exists in this area. However, the program has indicated that the area is far from homogeneous, and that major technical difficulties would require solutions before the sand, if present, could be utilised.

All these surveys were tied to the Australian Mapping Grid, and accurate position fixing was a relatively simple matter, achieved with the assistance of surveyors from the Department of Lands.

Outer Harbor Sand Survey

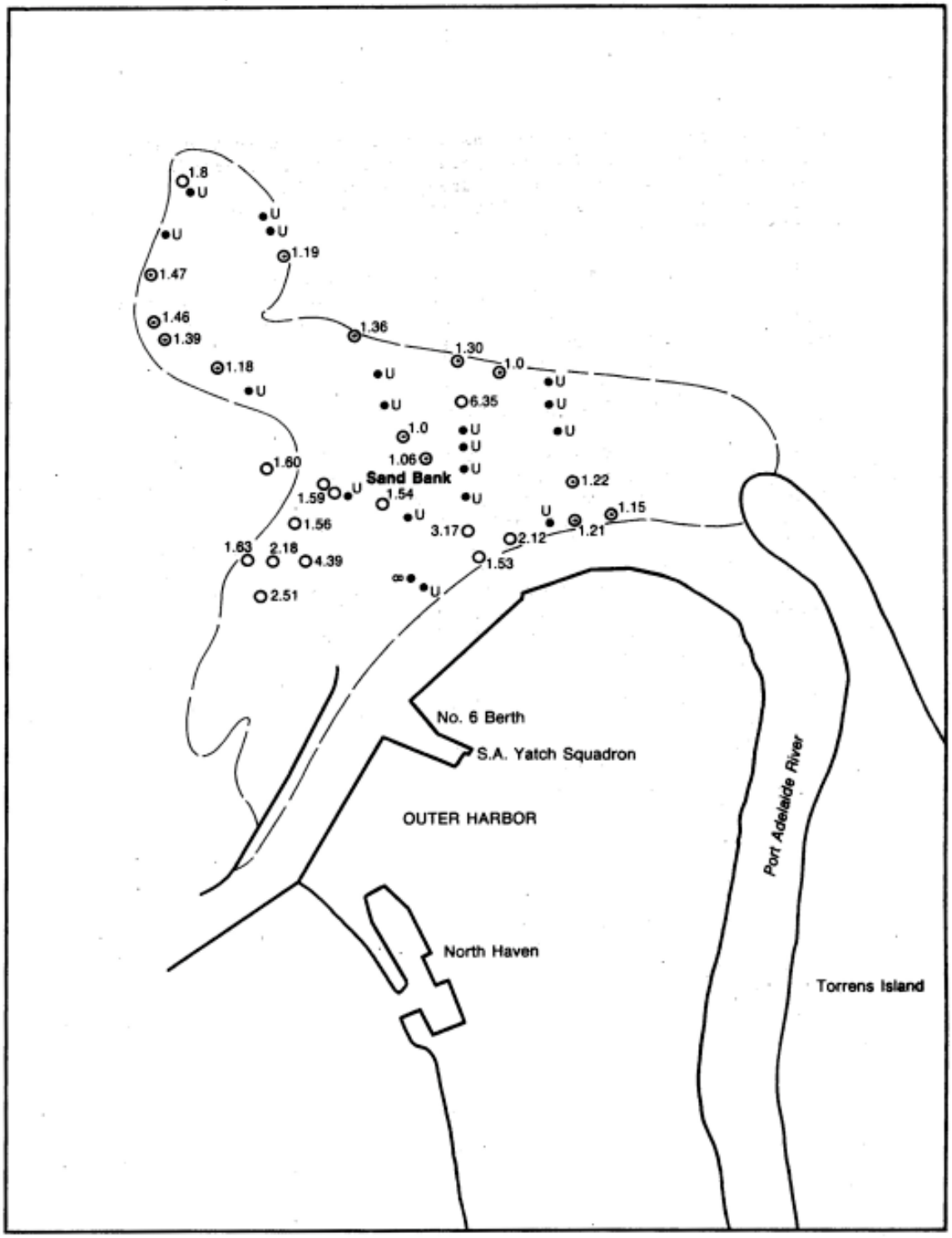
In late 1979, the Coastal Management Branch undertook a survey of the sand deposit immediately north of Outer Harbor, with a view to determining whether or not sufficient material suitable for beach replenishment could be obtained from this source. This survey followed initial exploratory investigation of the area earlier that year, and testing of sand that DMH had been dredging in nearby parts of the Port River. The latter material, which was being used for fill, was too fine to be considered for replenishment purposes. Evidence suggests that other dredged material from the river is likely to be even finer.

47 sample cores were taken as shown in Figure 8. The samples were obtained using a portable vibrocorer hired from the Flinders Institute of Atmospheric and Marine Sciences. This was operated from a small hired pontoon. The equipment was capable of obtaining samples to a 5 m depth of material, and most of the samples were to this depth. Grading, shell fraction and organic content were determined by AMDEL (Australian Mineral and Development Laboratories) for the Branch. The core samples were marked, as they were obtained, in 1 m depth sections, and these sections were analysed separately. Size analysis was done for each 1 m section of all cores; shell and organic content was determined for each 1 m section of four selected cores. Position fixing was by use of a sextant, with angles being taken off navigation channel markers and other known fixed points.

Because of problems with handling and fixing the pontoon in position in deeper water, sampling was mostly in shallow water of between wading depth and up to 4 m. The small area of dry sandbank was not sampled. Unfortunately, the limitation of the survey to shallow water has, in retrospect, reduced its usefulness. This is because the more economical dredging methods, such as using a trailing suction hopper dredge, cannot gain access to or operate in the areas sampled. However, the information is relevant to dredging from the southern, river side of the deposit using a bucket dredge or a cutter-suction dredge, which are options considered in this review.

A complementary seismic survey was considered, but was deferred pending consideration of the other information. Any further investigation of this sand source should include such a survey to determine the occurrence of the likely underlying calcrete layer. Experience gained by DMH in dredging the nearby channel indicates that the calcrete horizon is likely to occur approximately 6 to 6.5 m below the surface of the sandbanks sampled.

FIGURE 8 OUTER HARBOR SAND SURVEY – CORE LOCATION AND CALCULATED OVERFILL RATIOS



⊙ Rø crit. 1.0 to 1.5

O Rø crit. above 1.5

•U Unsuitable

0 1km

The analyses have been assessed to determine median grain size, both for each 1 m of core depth and for the total core at each sample location. Overfill ratios were calculated using the methods of Krumbein and James (1965) and James (1975) (CERC 1977). In better (coarser) parts of the deposit, they compare with results obtained for sand from some of the beaches being used in the present annual beach replenishment program. However, they do indicate that much of the deposit is unsuitable, and that most of it is marginal. The quantities in the general area are considerable, and the results suggest that, notwithstanding the unsuitability of much of the material, it may be possible to find enough suitable material.

The median grain size comparison is more favourable, with a high proportion of the samples being in the range 0.18 to 0.20 mm, as compared to the median grain size of the metropolitan beaches of 0.20 mm. There was some variation of median grain size with depth, though there was no consistent trend and, with the exception of a few cores, the variation was not extreme. The comments on this latter aspect have, however, been made on the basis of only a cursory examination, and the data would need to be further assessed and extended before the source were used. Shell content was in the range of 3% to 30%, though these are extreme values and 5% to 14% was more common. Although quite variable, as might be expected in a deposit of this type, most of the values are close to the 9% average value for the metropolitan beaches (Culver 1970).

The percentage by weight of total organic carbon, for the 14 cores tested, varied from almost none to 0.41% with an extreme value of 4.5% being measured. Most samples were in the range 0.1% to 0.35%. Visual examination of the cores during sampling indicated that the sand was reasonably 'clean' and that it would be unlikely to be offensive for use as replenishment material.

The results indicate that the deposit, if worked selectively, could provide an adequate replenishment material, but that excess amounts of between 20% and 40% might be needed to offset losses of fine material from the replenishment beaches. This imposes an obvious financial disadvantage. It should be noted, however, that the calculation of overfill ratios is not entirely reliable and should be used to consider the suitability of the sand in general, rather than to anticipate precise replenishment quantities. It should also be noted that the method of calculating overfill ratios makes no allowance for differences in shell content, though it is likely that more of the lighter shell fraction would be lost from a replenished beach.

Gulf St Vincent – Nearshore Sediment Dynamics and Sedimentation Study

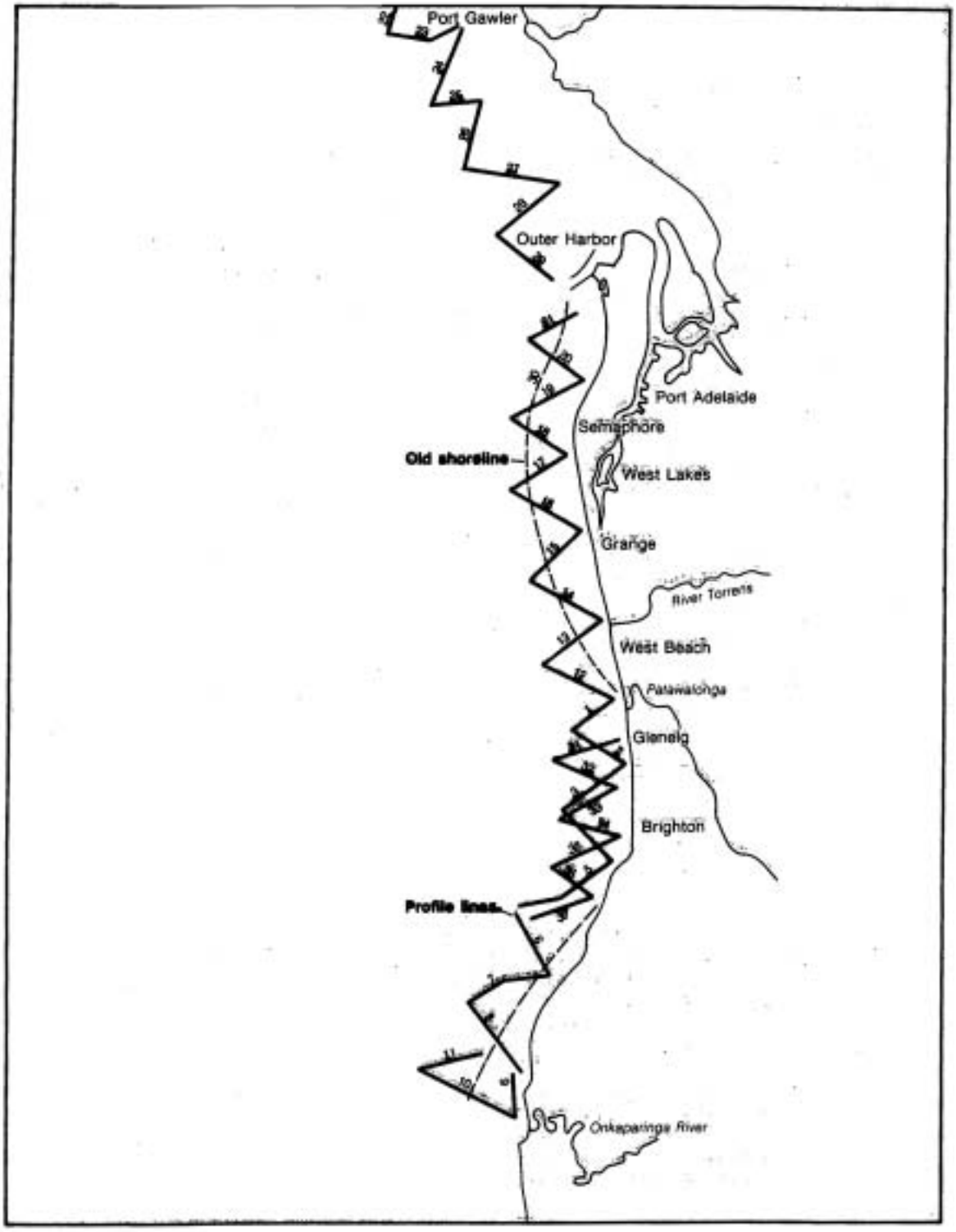
In early 1980, the Board commissioned an inter-university study of the seabed in the Adelaide region between Port Gawler and the Onkaparinga River.

An understanding of the behaviour of a coast cannot be divorced from a knowledge of the nearshore processes; and sediment characteristics, together with bedforms, can give valuable information about sediment movement and origin. Similarly, geophysical information, showing buried coastal cliffs and other hidden features, provides clues to historic processes and can thus shed light on present ones. The purpose of the study was to obtain and process this information, to extend the search for a possible offshore replenishment sand source, and to provide baseline information from which future changes might be measured.

The project was carried out jointly by the Centre for Environmental Studies and the Department of Geology, University of Adelaide, and the School of Earth Sciences, Flinders University, with Professor Sargent of the Department of Geology and Mineralogy, University of Queensland, undertaking the geophysical survey, as well as acting as a consultant.

The report (Hails et al 1982) is a lengthy one, containing detailed discussion and deductions from the survey data, and it is not practical to provide more than a summary here. It is also difficult to cover all the results of the study, because of the necessarily speculative nature of some of them (on account of limited evidence), and the hazard of quoting conclusions without all the supporting, and sometimes qualifying, discussion.

FIGURE 9 GULF ST VINCENT STUDY – KEY PLAN (HAILS ET AL 1983)



0 5km

The survey work was carried out in May 1980, using the 6.4 m Flinders University survey vessel equipped with echo sounder, seismic, and University of Queensland side-scan sonar equipment. The side-scan sonar provides an oblique 'view' of the seabed on either side of the vessel's course, and thus enables bottom features such as sand waves or ripples to be identified.

A 159 km cruise track, along 37 lines, as shown in Figure 9, covered a total area of approximately 480 square kilometres. 180 surface sediment samples were obtained from stations along the survey lines, using a small bucket dredger and a grab sampler. Washed split portions of these were sieved, and the various grain size parameters calculated from the sieving analysis.

Two aspects of the original proposal were deleted due to time and equipment constraints, and this affects interpretation of the results. Vibrocoring had been intended to ground-truth the seismic information, as had a check on the precision of vertical measurement (echo sounding), by repeated measurement on a placed concrete slab. The latter is of particular importance having regard to the possible significance of deduction based on the depth measurements.

Bathometric Changes. The echo soundings, after tidal adjustment, were compared with hydrographic information (Royal Australian Navy surveys) from 1946, 1948, 1957 and 1958. The areas of erosion and accretion were mapped (not included here), and volumes calculated. The measured variation was in the range between an accretion rate of 6.7 mm a year and an erosion rate of 1.97 mm a year. The net rate of change, calculated for the whole area using separately the NE–SW survey lines and the NW–SE lines, was 982,410 and 701,569 m^3 a year respectively – indicating an accretion rate of approximately 2 mm a year over the floor of the area investigated.

It must be noted that this assessment assumes a sufficient order of accuracy of both the original surveys and the present one, an assumption that may not be justified. The RAN hydrographic charts provide depths to the nearest foot in shallower water, and to the nearest fathom in deeper water, and the present survey did not include a precise check on depth measurement. Notwithstanding this, the authors consider that the trends were consistent and support the general finding. However, it must be borne in mind that the hypothesis relies on measuring an average change in seabed level of less than 7 cm, which, given the limitations of the data and the methods, must contain a high degree of uncertainty.

The comparison indicated large changes in the southern region, with the highest accretional rate off the Onkaparinga River mouth, and the highest erosional rate off Port Stanvac. Changes north of Outer Harbor were small and accretional. The region between Marino Rocks and the River Torrens mouth is described as characterised by successive belt-like areas of erosion and others of accretion. Erosion was found to be dominant further north, between Glenelg and Henley Beach, north of which an accretional trend was apparent.

The profiles showed a variety of seabed types – gently sloping, smooth seafloor, flat bottom with occasional outcrops, steep slopes with intermediate terraces, and other variations including several dune and sandwave features. These were grouped into eight categories, not all of which can be discussed here. The profiles between Semaphore and the Patawalonga showed two regions of steep slopes each following a terrace (thought to be related to sea level changes in the last 3,500 years). Those profiles between the Patawalonga and Marino Rocks showed an irregular bottom with a moderate slope. Composite sandwave or dune systems are the predominant bedform in this sector.

Sediment Parameters. The larger part of the data analysis was in the interpretation of the sieving results, and use of statistical parameters of these to deduce sources and movement of material. The parameters used are mean grain size, sorting, skewness, and the kurtosis.

Sorting is a measure of the degree to which the material has been sorted by natural processes into a limited range of grain sizes, either by selective transport of these sizes or by winnowing out of others. It measures the same characteristic as the engineering 'grading', but in the opposite sense. A 'well sorted' material is 'poorly graded'.

FIGURE 10 GULF ST VINCENT STUDY (HAILS ET AL 1983)

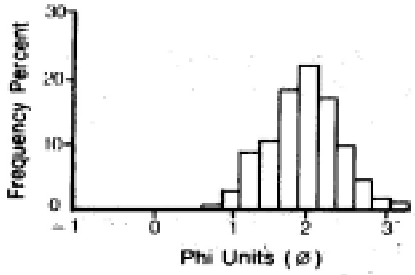


Figure 10A
Typical Uni-Modal Histogram
(Line 3 Sample 18)

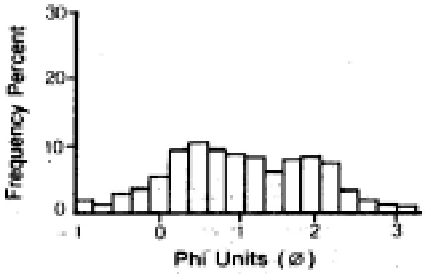


Figure 10B
Typical Bi-Modal Histogram
(Line 3 Sample 22)

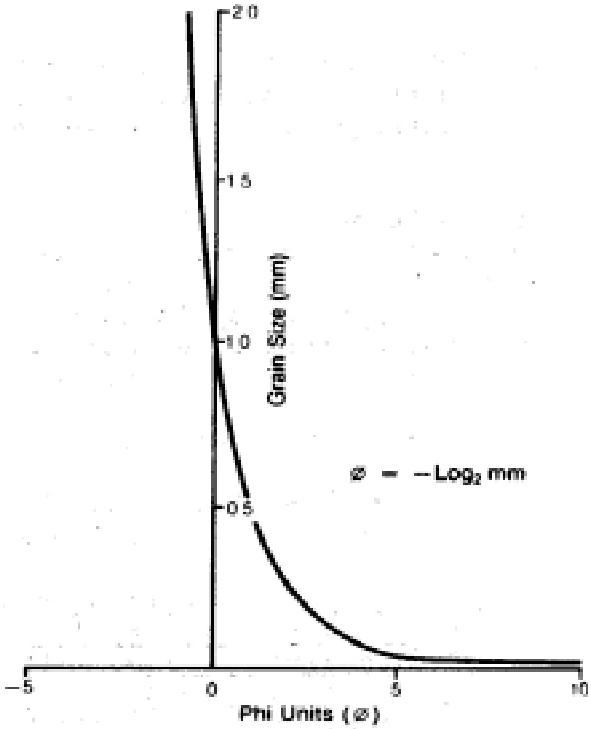


Figure 10C
Grain Size Units, Ø vs mm

Skewness is a measure of the asymmetry of the grain size histogram (see Figure 10), and indicates whether the material is predominantly finer or coarser than the mean.

Kurtosis is a measure of the 'peakedness' of the histogram, and can provide further information to enable correlation and identification of materials.

The sorting is measured by the standard deviation from the mean. The skewness and kurtosis are respectively the third and fourth moment measures, and can be calculated mathematically or by graphical approximations.

The skewness is a particularly useful measure, being sensitive to the environment. A negative skewness is found in materials laid down by a fluid flow strong enough to separate and remove the finer particles. It can also be indicative of an erosional trend. Positive skewness is associated with sediments laid down in a unidirectional current regime or in the absence of currents, and with accretion.

Changes in these parameters, from one part of the seafloor to an adjacent part, can be used to give possible indications of direction of the sediment movement and of the sea currents causing this movement. It can also be used to deduce which areas are contributing sediment (sources) and those into which sediment is moving and remaining in (sinks). It must be emphasised, however, that such deduction cannot be too heavily relied on, but should rather be considered as a valuable and potentially useful set of clues. There are factors that can result in misleading information. For example, relict material such as from a submerged coastline can, if recently exposed on the seabed, have parameters related to historic rather than present processes.

Figures quoted in the following text and figures refer in places to phi grain size units (Ø). These logarithmic-based units are now widely used and are more convenient for analysis of sand samples

according to the aforementioned criteria. They have been otherwise avoided in this report to avoid confusing the non-scientific reader. The relationship is $Ø = -\log^2$ (size in mm). The relationship is shown graphically in Figure 10C. It should be noted that high numbers indicate small grain sizes, and vice versa.

The main thrust of the sediment analysis was to postulate the sink/source distribution and the current movement pattern shown in Figure 12.* Examination of the seabed forms provided additional evidence to assist the University study team in deriving its postulated sediment and current movement diagram.

As shown in the figure, the deduced sediment circulation has the following features:

- The Onkaparinga River to O'Sullivan Beach, Largs Bay, and Outer Harbor to Port Gawler sectors are sinks rather than sources. It is thought that they receive sediments from various sources.
- The O'Sullivan Beach to Hallett Cove sector and the coastal cliffs behind are only a minor source of sediments.
- The Hallett Cove to Marino Rocks sector could be a source or a sink depending largely upon the direction and magnitude of currents, and possibly on wave energy concentration.
- The Marino Rocks to Semaphore Beach sector seems to receive small quantities of sediments from offshore, most of which are moved northward in the nearshore zone and deposited south of Outer Harbor.

It should be noted that these deductions are based on sparse sampling of material on the surface of the seabed and on unverified remote sensing of the bedforms. They are unproven and must remain hypothetical until either supported by further evidence or disproved. Although the evidence from the different methods (sediment sampling and bedforms) is corroborative for some parts of the study area, it is in conflict within itself at others, and is also not entirely in accord with known or likely current movements.

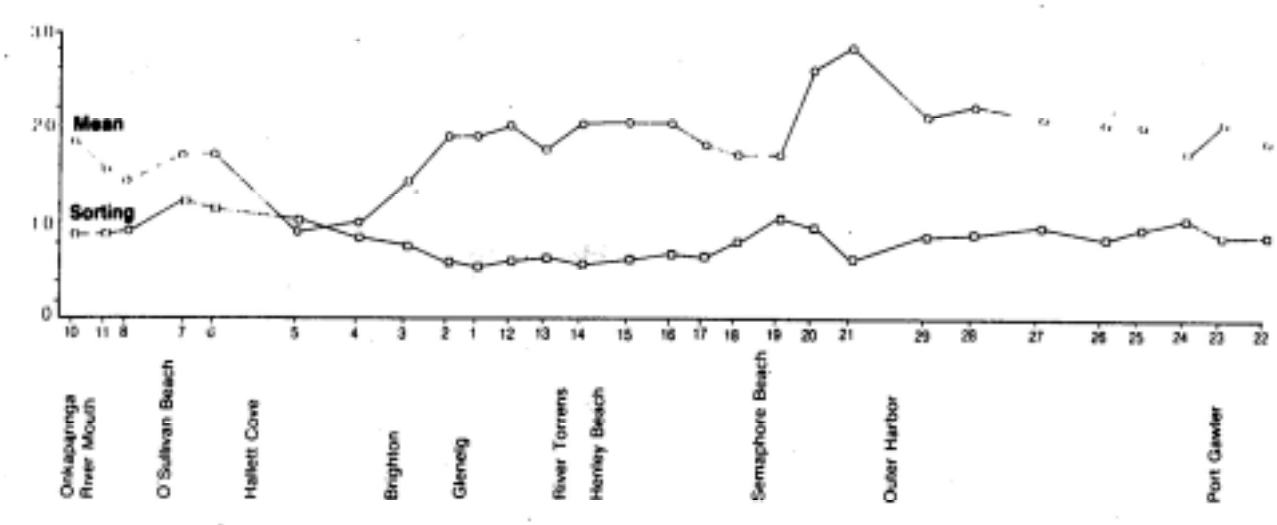
The general description of the sediments in the study area, as quoted in the study report, is as follows:

> Sediments in the area between Outer Harbor and Port Gawler are medium to fine sands. Their pattern of distribution appears as lobes of fine sand parallel with the trend of bathometric contours. Medium sand is concentrated in the central part of the vicinity of the Gawler River mouth. From Outer Harbor southward, sediment is graded from very fine to very coarse and exhibits a more complex distribution pattern. On the southern side of Outer Harbor, where Largs Bay is situated, fine sand has accumulated. On average, this sand is the finest material found along the study area (Figure 11A*). Within Largs Bay, the sand fines northward and seaward. The geometry of the coastline together with the Outer Harbor wave-breaker certainly modify the current regime in the Bay resulting in the trapping of this fine material. The primary feature of mean grain size distribution south of Largs Bay to Henley Beach is the lobe of medium sand that extends to a north-west direction from Port Malcolm seaward. Farther south towards Brighton the nearshore material is fine and becomes coarser offshore. Figure 11B* shows the gradient of mean grain size with depth of 2-m intervals along the study area. Coarse sand is concentrated between Brighton and Hallett Cove where two coarse sand bands are extending seaward in north-west and south-west directions. In between these two bands, mean grain size decreases seaward, a feature that was also observed in Largs Bay sediments. Extreme low and high values of mean grain size are found in the area between Hallett Cove and the Onkaparinga River mouth (Figure 11B*) where small patches of very coarse and very fine sand occur. The local effect of the Noarlunga reef is observed in the mean grain size distribution in this area.

* The figure numbers have been altered to suit the number of those reproduced here.

FIGURE 11 GULF ST VINCENT STUDY (HAILS ET AL 1983)

Figure 11A Distribution of Average Mean (Ø) and Sorting (Ø)



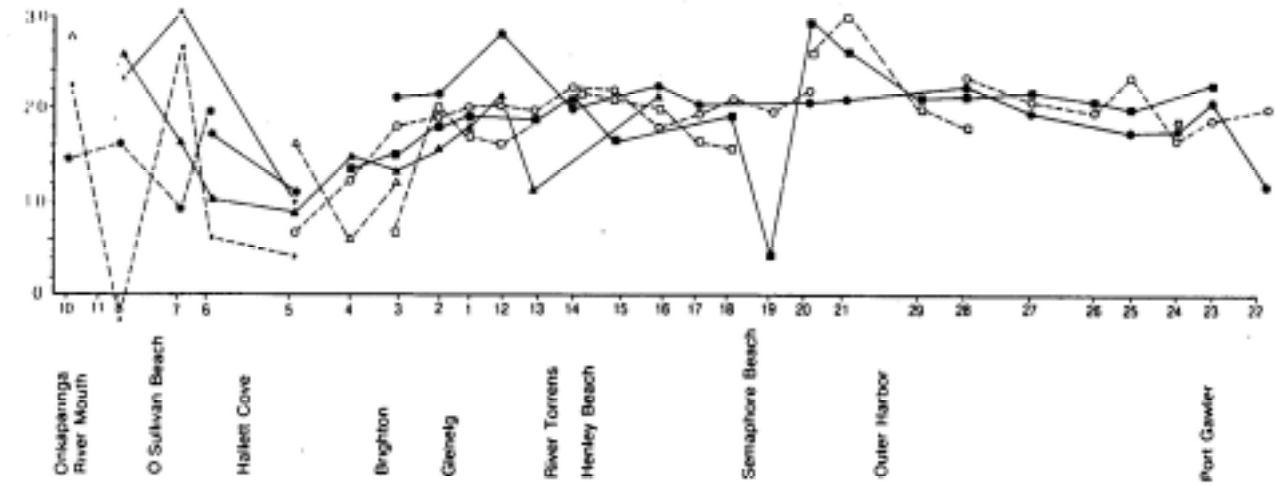


Figure 11B Distribution of Mean Grain Size at 2-Metre Depth Intervals

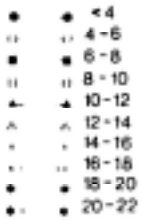
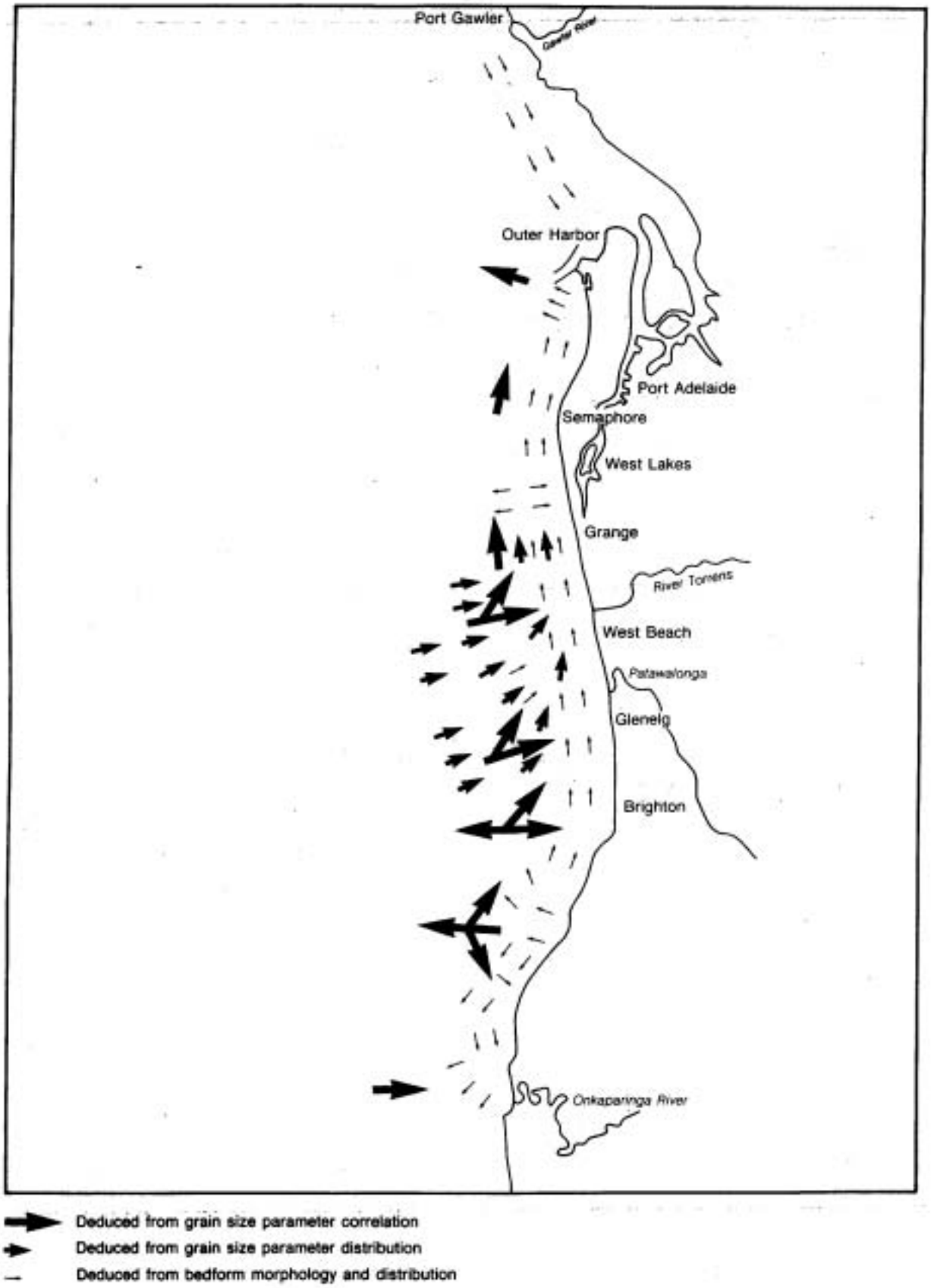


FIGURE 12 GULF ST VINCENT SEDIMENT STUDY DEDUCED SEDIMENT MOVEMENTS AND BOTTOM CURRENTS (BASED ON HAILS ET AL 1983)



Other findings of the study, based on the sediment parameters, are as follows:

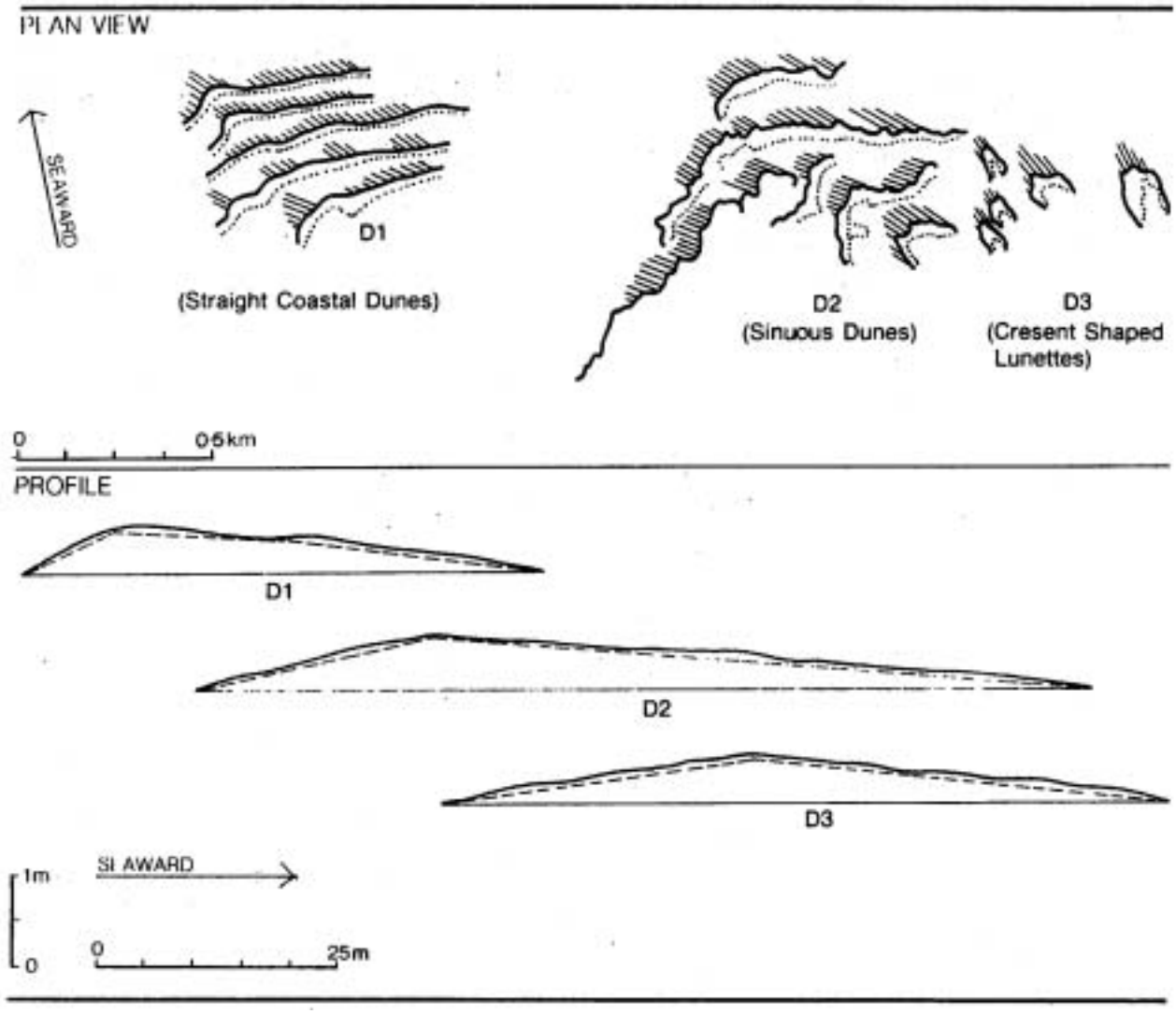
1. The average of the mean grain size for the surface sediments in the study area is 1.79 Ø (0.29 mm) with extremes at –0.53 Ø (1.43 mm), or very coarse sand, to 3.02 Ø (0.12 mm), or very fine sand. Medium and fine sand samples form 54.02% and 35.06% respectively of the samples, whereas very coarse and very fine sand samples constitute only 1.1%.
2. Most of the sediments are polymodal, particularly those from north of Outer Harbor and south of Brighton, a fact that indicates either different sources of material or various levels of energy. At Outer Harbor, this is confirmed by a change in carbonate content. The likelihood of sand entering this area from both alongshore and offshore is discussed in Section 2.6.3. For bi-modal sediments, examination shows that the coarser fraction is mostly calcareous material. This applies mainly to samples from north of Outer Harbor. Sediments from south of Brighton have the secondary mode (the larger grain sizes) mostly of terrigenous sand.
3. In general, sand is finer northward and seaward. The coarsest sand is concentrated between Brighton and Hallett Cove, whereas the finest material is found just south of Outer Harbor.
4. Sediments are predominantly moderately to poorly sorted. Sorting shows a similar pattern of distribution to that for mean grain size, and is generally poorer northward and southward from the Glenelg – Henley Beach area.
5. The majority of surface sediments are negatively skewed, which indicates the abundance of coarser material.
6. Extreme values of mean grain size, standard deviation, skewness and kurtosis are found in sediments from the Hallett Cove to O'Sullivan Beach area. These values are thought to reflect the higher level of wave energy that prevails in this area.
7. The study area does not behave as one unit that responds in the same way to the hydrodynamic forces. It is best described as a series of coastal sectors (or cells), based on both the morphology and sediment characteristics. Seven sectors are identified that have the same topographic and sediment characteristics. Grain size and sorting, and the correlation of these with water depth varies from one sector to another.
8. Examination of mean grain size of all the samples in relation to water depth does not reveal a clear relationship, though there is an indication that most surface sediments from shallow water (< 10 m) and from deep water (> 20 m) are near the grade of fine sand, while those from intermediate depths tend to be coarser. Examination by sectors shows good correlation for some sectors, the Onkaparinga River mouth to O'Sullivan's Beach sector showing the best correlation, with the coarser material seawards. Good correlation was also obtained at Largs Bay, but with the material becoming finer seawards. See Figure 2.10B.
9. A similar study of sorting in relation to depth showed little correlation except that sorting improves slightly as water depth increases.

Note: Some of these findings, and those to follow in the next two subsections, have been quoted directly from the conclusions of the study report. Others have been extracted from other parts of the text. Not all the conclusions of the study are presented here.

Bedforms. Bedforms were assessed using the sonograph records, appropriately corrected for distortion, and using colour aerial photographs to supplement these.

Three bedforms, all of a 'current transverse' type, were identified in the study area, these being sand dunes, which are dominant, ripples, and sandwaves, which occur only in special areas. They are shown in Figure 13. Another, larger bedform, described as sand ribbons was identified from the photography in deeper water outside the study area. These sand ribbons, of length exceeding 2 km and width exceeding 200 m, appear to be 'current parallel' and at right angles to the underwater sand dunes they meet where they terminate.

FIGURE 13 GULF ST VINCENT STUDY BEDFORM TYPES (HAILS ET AL 1983)



The sand dunes (of height approximately 0.5 m and wave length 50 to 150 m) occur between Marino Rocks and Henley Beach, and are best developed at water depths between 7 and 8 m. Three types were identified, as illustrated in Figure 13. The longer straight crested type (D1) occur between Marino Rocks and Glenelg. These are asymmetric in profile with the lee side facing onshore, suggesting the domination of landward currents and landward sediment movement.

Further north, between Glenelg and the Torrens River, the sand dunes (type D2) are more complex with strongly curved to sinuous crests, and although of similar wave length to type D1, have almost the same height. Their asymmetry again indicates that landward currents are dominant, and that these have a tendency to deviate towards the north or south.

Further north the dunes become smaller and their shape changes – to crescent shaped lunettes, in the form of isolated dunes, with a slightly asymmetric profile (D3). Their asymmetry shows that landward flow is still relatively strong, but also that the return flow is significant. Farther north they become smaller and less frequent, and in some the horns become joined, providing additional evidence of return flow.

The report compares the regional distribution of these dunes with the deduced bathometric changes (as obtained by comparison of the echo soundings with early hydrographic charts), and reports that the longitudinal dunes (D1) exist in a predominantly accretional zone, the curved and sinuous dunes (D2) in areas of both accretion and erosion, and the lunette dunes (D3) in a

predominantly erosional zone. It should be noted that these do not refer to the coastlines adjacent to these areas, these coastlines showing distinctly different erosion/accretion characteristics.

Other deductions or observations, made from the side-scan sonar records and field observations, are that the growth of seagrass plays a significant role in stabilising bedforms. The comment is made that these bedforms appear to be relict ones, which have been sustained by a continuous supply of sand and by the seagrass growth. It was suggested that the sand ribbons indicate a limited offshore supply of sand. Derivation of the sediment movement diagram, by combined use of the side-scan sonar and sediment characteristics has already been discussed.

Geophysical Results

The geophysical, seismic survey was intended to detect and measure layers of unconsolidated material, with a view to determining areas of accumulation. It was also intended to locate possible old shoreline positions or other features that would provide information on the sea level changes and their effects. Typical seismic profiles are shown in Figure 14. The main areas of accumulation were found to be off the Onkaparinga River mouth, where recent accumulations of greater than 5 m thickness were found, and north of Outer Harbor, where thickness was in the range 7 to 11 m.

A likely ancient shoreline was identified between Glenelg Beach and Outer Harbor approximately 3 km offshore, and a northwards-dipping hard rock basement was identified between the Onkaparinga River and Hallett Cove. Other horizons were also found and will be discussed in the following text. The lack of an identified horizon between Marino Rocks and Glenelg is notable.

Difficulty was found in correlating reflectors from profile to profile over the entire study area, which was therefore divided into 4 areas of similar reflector characteristics. The findings for these areas are summarised under the separate area headings.

Area 1 – Onkaparinga Mouth to Marino Rocks (profile lines 4 to 8 and 10)

This area is characterised by a hard rock basement with more than 7 m of sediment (increasing thickness seawards) off the Onkaparinga River. The thickness of sediment was found to decrease northward to less than 0.5 m in places off Hallett Cove.

This supports the findings, based on sediment characteristics, that the area off the Onkaparinga River is accretional, and that the area off Hallett Cove may be erosional.

The seismic records also showed many old channels, since in-filled by sediment, off the Onkaparinga mouth. One of these is more than 30 m deep and about 300 m wide.

The report suggests that the hard rock basement in this area is an offshore extension of the regional geologic structure, the Eden Block.

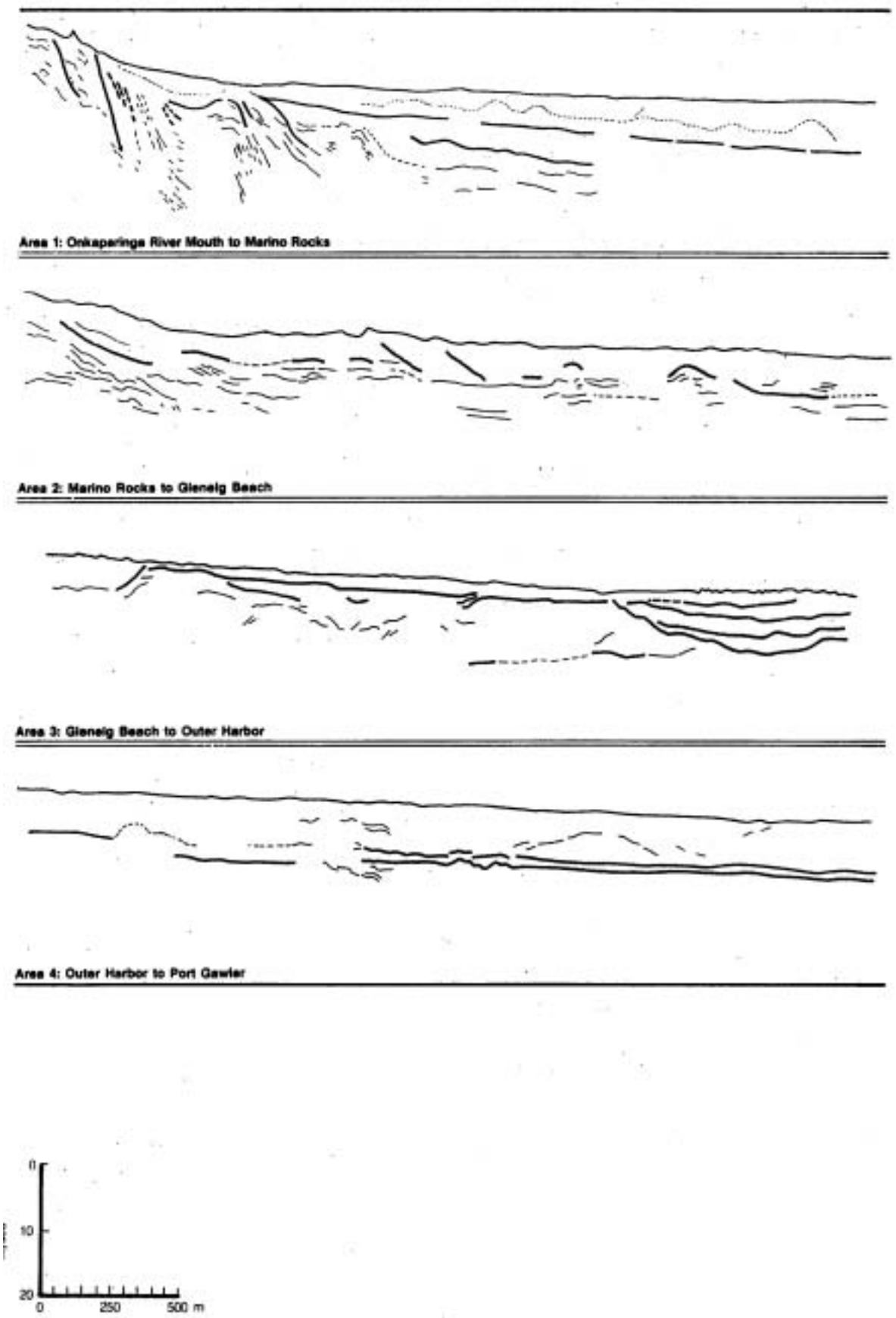
Area 2 – Marino Rocks to Glenelg Beach (profile lines 1 to 4 and 31 to 35)

Poor seismic reflection profiles were obtained from this area, allowing little interpretation other than that there may not be a reflector horizon. The lack of reflectors is reported to be either due to possible massive seagrass growth or to ‘the type arid morphology of the seafloor’. The lack of a horizon is surprising, as the ‘Laser Survey’ described earlier in this section showed extensive near-surface calcrete layers similar to those that occur further north. The latter calcrete layers are clearly and consistently identified in the present seismic records.

Area 3 – Glenelg Beach to Outer Harbor (profile lines 12 to 21)

One horizon was found to occur consistently, and to decline sharply seawards at a distance of approximately 3 km from the coastline. This steep slope, which occurs along a line approximately parallel to the present coast, is thought to represent an ancient shoreline. Landward of this the horizon is relatively flat and overlain by less than a metre of sediments. Previous coring and outfall construction off Port Malcolm shows that this horizon is Pleistocene limestone or calcrete.

FIGURE 14 GULF ST VINCENT STUDY TYPICAL SEISMIC PROFILES (HAILS ET AL 1983)



Seaward of the declination thicker layers of sediments occur, with three reflector horizons being recorded. These are thought to be likely to represent three successive stages of sedimentation and calcrete formation during the Pleistocene. Sediment thickness above the uppermost of these horizons varies from 1.3 m on line 13 to a maximum of 10.2 m on line 16.

Area 4 – Outer Harbor to Port Gawler (lines 22 to 29)

A smooth, linear and continuous single horizon was found at between 7 and 11 m below the seabed, and is thought to be a calcrete layer separating the Holocene and Pleistocene deposits. This is confirmed at Outer Harbor by DMH dredging. A calcrete layer, overlying stiff clays, has been found at a depth of approximately 6 to 6.5 m below the nearshore sand banks (low tide level approximately).

Summary – Further Exploration Needed

1. No practical replenishment sand source has been located off the southern part of the study area, despite at least one early promising sample. The likelihood of a large enough source being found now seems to be too small to warrant further expenditure on searching this area.
2. Other studies, including the Gulf St Vincent Nearshore Sedimentation Study, have not definitely revealed any other source, but have suggested possibilities off the Onkaparinga River, possibly in the Marino Rocks – Hallett Cove area, and possibly off the northern part of the study area (Wonga shoal). It is notable that there is a tendency to finer sand seawards, and that the nearshore zone off the Onkaparinga is an exception to this – at least for surface sediment.
3. A deposit that is useable, but not ideal, has been established off Outer Harbor, but the area proved is in water too shallow for operation of the most likely type of dredge.
4. This review has shown that a major replenishment is practical and not economically out of the question (in comparison to the present policy). It is therefore important to extend the search for offshore deposits, and to prove a more suitable area than that at Outer Harbor. Initial further exploration is recommended off the southern coast (especially off the Onkaparinga) and off the northern part of the study area (including the Wonga shoal). If this fails to identify a suitable source, the Outer Harbor area should be further sampled – in deeper water adjacent the area already proved. Every effort should be made to find a sand that is at least as coarse as the beach sand, and preferably coarser. This is desirable to reduce to an absolute minimum the risk associated with a major replenishment – usually that an excessive amount of sand will be lost either offshore or alongshore. Unsuccessful beach replenishment elsewhere has invariably been associated with use of sand that was too fine.

2.3 SEA LEVEL CHANGE

2.3.1 Background and Context

On a long-term and worldwide basis, changing sea level is undoubtedly the greatest single factor influencing coastal change. Its effect in establishing the Adelaide coastline has been discussed in Section 2.2.1. In the 50- to 100-year time spans relevant to planning and engineering studies, it can be significant, as is obvious in the case of the costly flood protection required for London, UK, the sinking of Venice, and the critical loss of beaches along the Atlantic coast of America. In these cases, the relative rise has been accentuated by simultaneous sinking of the land, as is thought to apply at Adelaide to a lesser degree. Future rapid sea level rise caused by warming or the earth due to increased carbon dioxide in the atmosphere (the 'greenhouse' effect) could become an over-riding factor (Hoffman et al, EPA, 1983). The US Environmental Protection Agency considers that the rise is likely to be between 1.44 m and 2.17 m by the year 2100.

At Port Adelaide the increase in sea level relative to the land is thought to be of the order of 1.5 to 2 mm a year, with a possible decrease southwards towards Brighton. This is equivalent to 9 to 10 m

of beach width a century, assuming an average beach slope at 1:40. The effect on the sediment budget is discussed in Section 2.6.3.

Glacial retreat has occurred over the past 100 years, and was generally thought (King 1972) to be the main reason for the present worldwide rise in sea level. Other factors, such as change in ocean temperature, continental drift, and isostatic seabed and coastal land level changes, were thought to apply to a lesser degree. (Isostatic changes are the response of the seabed and adjacent lands to pressure changes caused by change in sea level.) It has more recently been shown (Nummedal 1983) that the thermal expansion of the body of seawater itself may account for as much as half of the sea level rise. Nummedal's model showed sea level lagging the mean global temperature by 18 years.

A review of mean sea level change was carried out in the Branch as a part of the present study (Petrusevics 1982). Interpretation of the Port Adelaide and Outer Harbor tide gauge results was reviewed, and local trends compared with reported ones worldwide and elsewhere in Australia. An examination of tidal records between 1966 and 1978 was included to determine change during this period and to identify shorter-term periodicity. The following sections contain information mainly extracted from the Branch technical report.

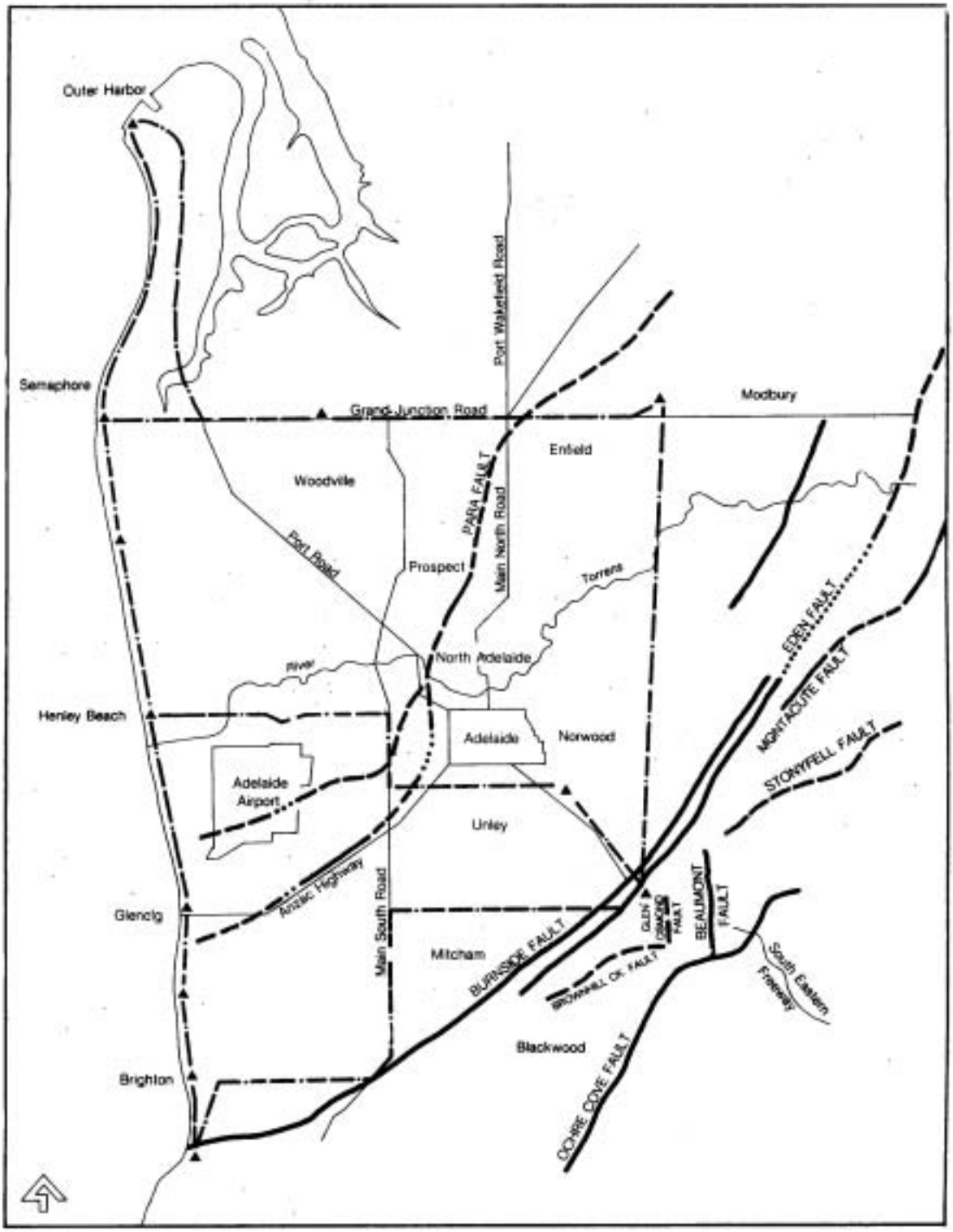
2.3.2 Adelaide Coastal Land Settlement

There is evidence that sinking of the coastal land accounts for a significant part of the relative change at Adelaide (Culver 1970). There could be several reasons for this. Following an enquiry by the Branch, the Department of Mines and Energy recently advised that it knew of 'no new data on which to base comments on the apparent rise in mean sea level relative to the land in a specific sense'. However, it suggested three mechanisms that were considered to have a possible bearing on settlement of the coastal land. These were tilting or other movement of fault blocks (fault locations are shown in Figure 3); draw-down of aquifers, resulting in consolidation; and long-term drying out and shrinking of expansive clays.

Culver compared a 1969 re-survey of early Engineering and Water Supply Department benchmarks with the original 1880 survey. He assumed 'stable' marks in the Hope Valley reservoir area. This area is closely underlain by solid rock forming part of the Mount Lofty ranges, and could be expected to be stable in relation to the Port Adelaide/Semaphore area, which is founded on soft sediments. He determined a settlement in the Port Adelaide/Semaphore area of approximately 0.6 feet (183 mm) a century. There was, however, a considerable scatter in the results, probably due to local movement caused by near-surface clay moisture changes. In addition, the survey was to 'third order' accuracy, which is barely adequate for this use. However, it was, and still is, the only information available. Its use did provide a rate of settlement that closely accounts for the difference between the apparent local sea level rise, as deduced by Culver, of 253 mm a century, and the generally accepted eustatic sea level rise of approximately 100 mm a century (King 1972).

The Branch recently initiated a project to determine a more reliable estimate of the settlement. 'Stable' benchmarks were placed, in autumn and winter 1982, on a survey network shown in Figure 15. The network was chosen to tie in to solid rock outcrops at three places, and also to cross the various faults at a sufficient number of places to enable any change across them to be detected. The benchmarks in soft sediments were placed to depths of between 10 and 17 m and were designed to minimise the influence of movement of near-surface clays. Holes were drilled, and cores retained for reference. This work was carried out for the Branch by the Department of Lands with assistance from the Department of Mines and Energy. The Department of Lands surveyed these marks in autumn and winter 1983 to a 'levelling of high precision' standard. This is the highest level of precision attainable. Repeat surveys are intended at 5-yearly intervals. It is anticipated that useful preliminary results will be available in 1988 and reliable results in 1992.

FIGURE 15 PRECISE LEVELLING SURVEY NETWORK (SURVEYED 1983)



FAULTS

- Accurate—from geophysical survey or field observation
- Inferred—requires checking by geophysics
- Conjectured—concealed by thick soil deposits
- ▲ Proposed new B.M. sites
- Approximate levelling network

2.3.3 Longer-Term Changes

Figure 16 from Curray (1965) shows sea level change during the past 30,000 years, as deduced from radiocarbon dating of deposits known to have been related to sea level. It is generally thought (Komar 1976 et al) that a rapid sea level rise of approximately 8 mm a year occurred during the early Holocene, between approximately 18,000 BP and 7,000 BP, and that the rate then reduced to about 1.4 mm a year until the sea reached near its present level approximately 2,000 to 4,000 BP. Figure 17 (from Shepard and Curray 1967) shows sea level for the past 8,000 years based on data from tectonically 'stable' parts of the world. Also shown in this figure is the curve of Fairbridge (1961), based on changes deduced from glacial advances and retreats. The latter suggests higher stands of sea level than the present.

FIGURE 16 SEA LEVEL CHANGE OVER PAST 30,000 YEARS (CURRAY 1965)

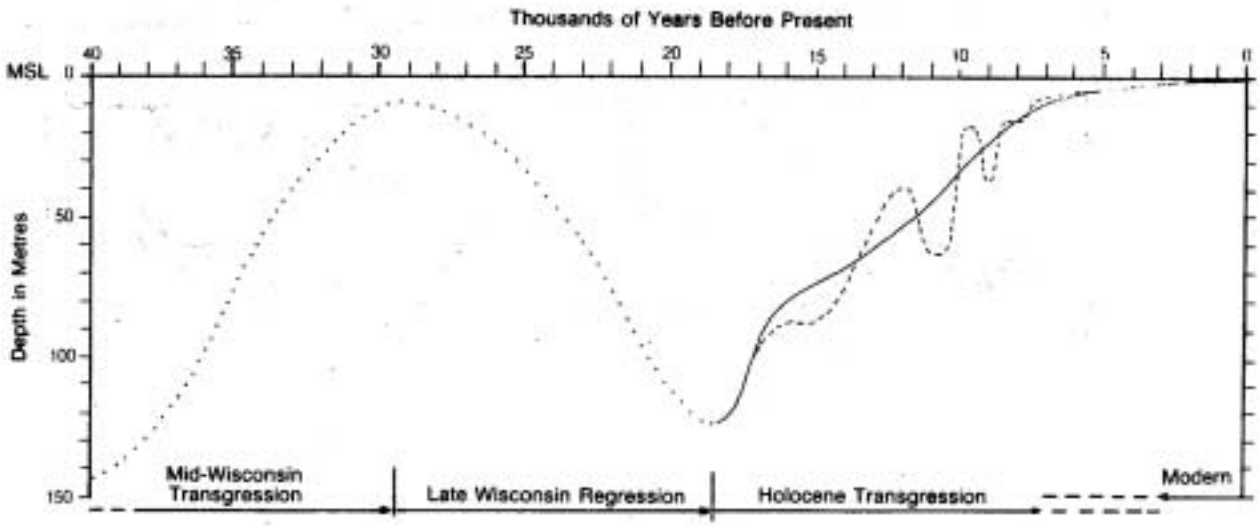
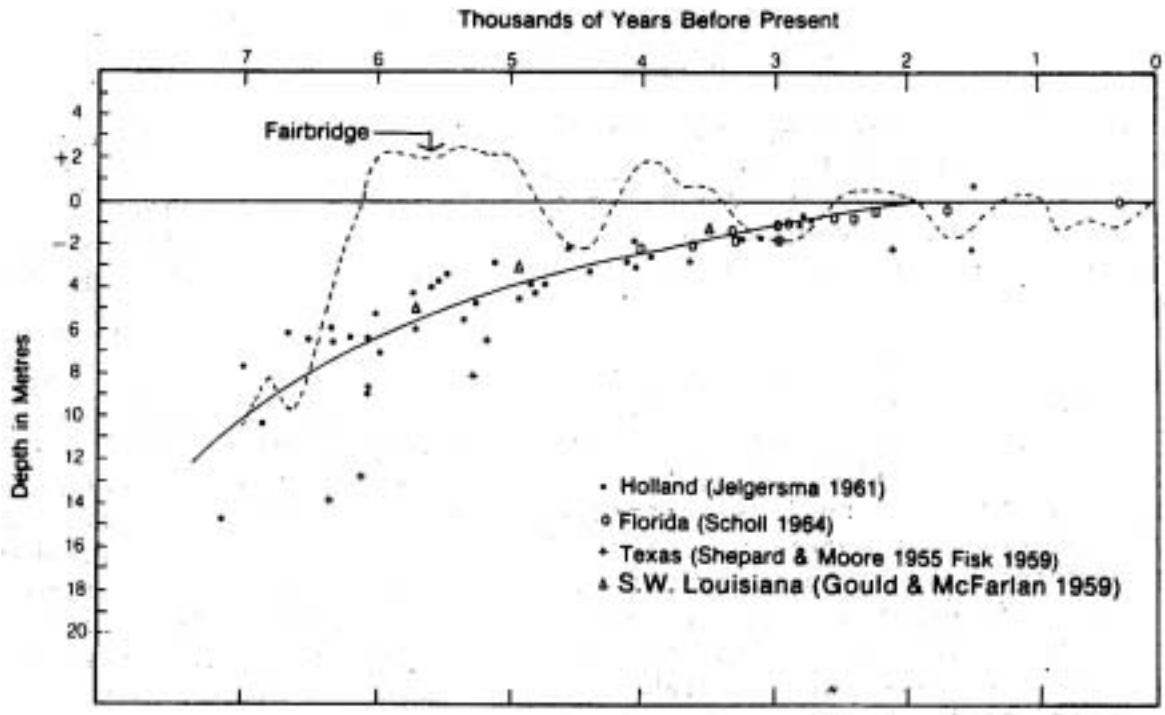


FIGURE 17 SEA LEVEL CHANGE OVER PAST 8,000 YEARS (SHEPHERD AND CURRAY 1967)

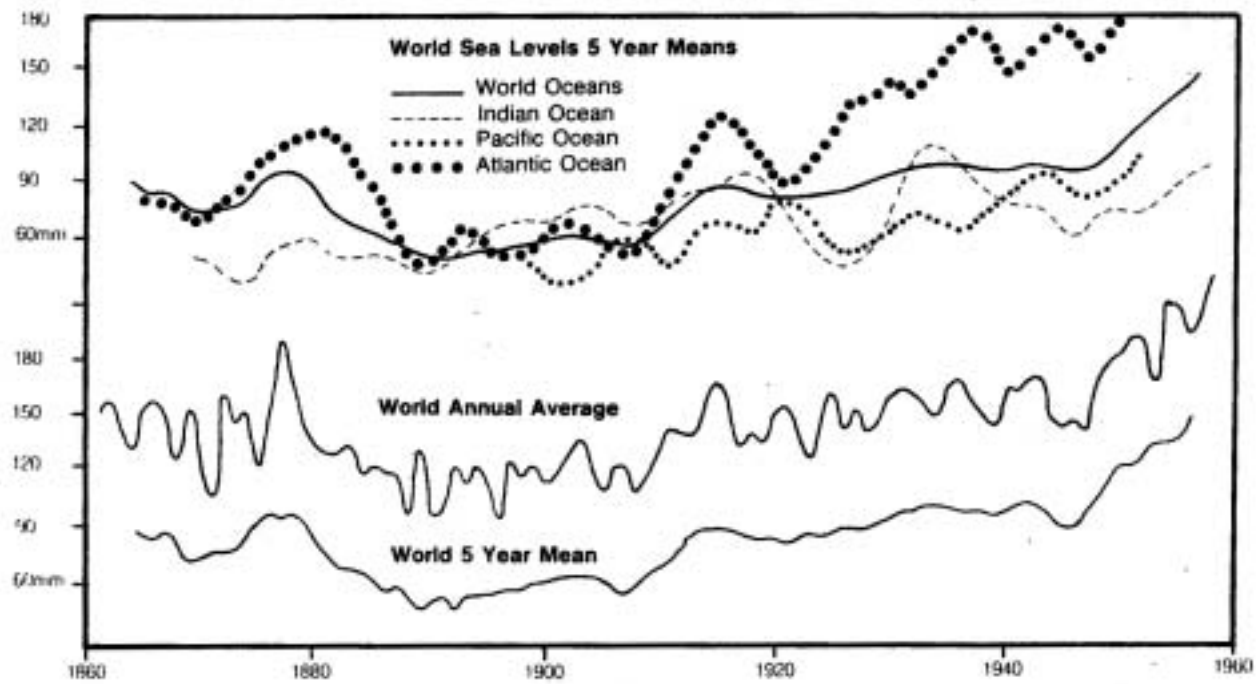


As discussed in Section 2.2.1, geological opinion has been divided both internationally and locally as to whether or not a sea level above the present one occurred during the mid Holocene period. Belperio et al (1982) argue that there is inadequate local evidence for this in South Australia. King (1972), in summarising findings on a worldwide basis, notes that evidence for higher sea levels, mainly from Australia and New Zealand, is from mountainous areas and is likely to be suspect because of tectonic land movements. He documents considerable support for the theory that sea level has risen slowly over the past 5,000 years, and that it is still rising.

2.3.4 More Recent Changes – The Past 100 Years

Published data, based on analysis of tide gauge records over the past 100 years or so, suggests that world sea level has increased at an average rate of just over 1 mm a year during this period. Figure 18 from Fairbridge and Krebs (1962) shows the trends for the various oceans and the world annual average up to 1960. As noted from the figure, there are significant shorter-term changes. The highest rate, during the period 1946 to 1956, was 5.5 mm a year. Published information about worldwide changes since 1960 is limited, though Hicks (1981) obtained an average of 1.3 mm a year between 1940 and 1978 for forty-nine North American stations. The largest and most recent survey is that by Gornitz et al (1982), which was based on results from 700 tide gauges around the world. After making adjustments for known land uplift or subsidence, they found that sea level is rising at a rate of 1.2 mm/year in all geographic regions.

FIGURE 18 WORLD MEAN SEA LEVEL, 1860–1960 (FAIRBRIDGE AND KREBS 1962)



Although the rise deduced from local tide gauge records seems to agree well with these worldwide trends, this is not necessarily meaningful. Most of the information has come from Northern Hemisphere ports, and the result is correspondingly weighted towards these. Professor Lennon of Flinders University cautions against any assumption that rates of change in the Northern Hemisphere will apply here (pers. comm., 1982). It must also be borne in mind that changes in eustatic sea level are extremely difficult, if not impossible, to measure with reliability because of changes in the reference land levels. Meteorological conditions such as, for example, a season with a higher than usual amount of onshore wind, can significantly influence the annual mean sea level at a particular place. This adds to the difficulty in trying to determine the real eustatic sea level rise. The various studies that have been referred to show poor correlation between ports, despite the selection of these to avoid those in areas of known tectonic instability.

The recent Branch review of sea level change (Petrusevics 1982) found only one Australian study of long-term recorded change. Foster and Stone (1965) examined sea level records at Fort Denison, Sydney Harbour, for the period 1897 to 1963, and found no marked change in sea level over the total period, though increases and decreases at rates of 5.8 and 3.8 mm a year respectively were noted. The authors, in relating the sea level change to erosion of Cronulla Beach, made the interesting observation that there was no relationship between increasing sea level and erosion, but rather that the reverse had occurred. The beaches had eroded during a period of no significant sea level change, and had accreted during a period of sea level rise.

Tide gauges have been in operation at Inner Harbor, Port Adelaide, since 1882 and at Outer Harbor since 1943. Records from Port Adelaide were used by Culver (1970) to deduce the sea level change and these, together with those from Outer Harbor, were subsequently used by DMH in a more recent assessment of the change. While these gauges provide the only long-term local record of sea level change, their limitations must be recognised. There are large gaps in the record, as shown in Figure 19A and Figure 19B, and there is uncertainty about early datum changes. Also, the gauges are established and maintained primarily for port operation use and a high level of precision should not be presumed. Petrusevics (1982) noted a large and inexplicable difference in the rise in sea level between 1966 and 1980 as measured from these two gauges.

By fitting the best straight line to the Port Adelaide records (Figure 19A), Culver determined a rate of sea level rise of 2.4 mm/year. More recent information from DMH (Figure 19B) indicates a lower rate of change of approximately 1.5 mm/year for Port Adelaide, but a similar, higher rate of change for Outer Harbor, though the latter is based on only a short period of record. Unfortunately, the details of tide datum shifts are shrouded in history, and information that has come to light from DMH archives seems to have been unclear or inadvertently used in the wrong context.

Culver took account of a datum change of 0.28 feet in 1921, but there was apparently no datum change at Port Adelaide in that year, though gauges at Semaphore and Outer Harbor were corrected by the 0.28 feet in 1921 (Pickering, pers. comm., 1984). This adjustment should not have been made to the Inner Harbor records that Culver used. However, an earlier datum adjustment in 1907 may have been confused with the 1921 corrections and should have been included. Since this adjustment was 0.30 feet (ie very nearly equal to the 0.28 feet used by Culver) and was in the same direction, Culver's deductions should be very nearly correct. The date discrepancy is irrelevant because no records between the two dates were used.

The difference between Culver's and DMH values has not been accounted for, and perhaps will not be without extensive further search of DMH archives. It would appear, however, from examination of Figure 19B, that the higher rate of change has applied over the past 40 years at both Outer Harbor and Inner Harbor.

2.3.5 Short-Term Mean Sea Level Fluctuations

Short-term changes appear to occur as both large increases over periods of approximately 10 years and smaller fluctuations over an apparent period of 3 to 4 years.

A recent Flinders University study (Birch 1980) examined records from nine southern Australian stations, including Port Adelaide and three other South Australian ports, and reported an apparent mean sea level rise of 7.5 mm a year for the 10-year period from 1966 to 1975. Although this figure appears to be high in relation to the trend over the past 100 years, it is comparable with the worldwide rate of sea level rise of 5.5 mm a year between 1945 and 1955 (Fairbridge and Krebs 1962), and is apparently based on good correlation for the stations examined.

It is noteworthy that Foster and Stone (1965) detected a rate of sea level rise of 5.8 mm a year for Fort Denison, Sydney Harbour, for the 1945 to 1955 period, and that a similar rise of 6.8 mm was obtained from Outer Harbor records for the same period.

FIGURE 19 PORT ADELAIDE MEAN SEA LEVEL CHANGE

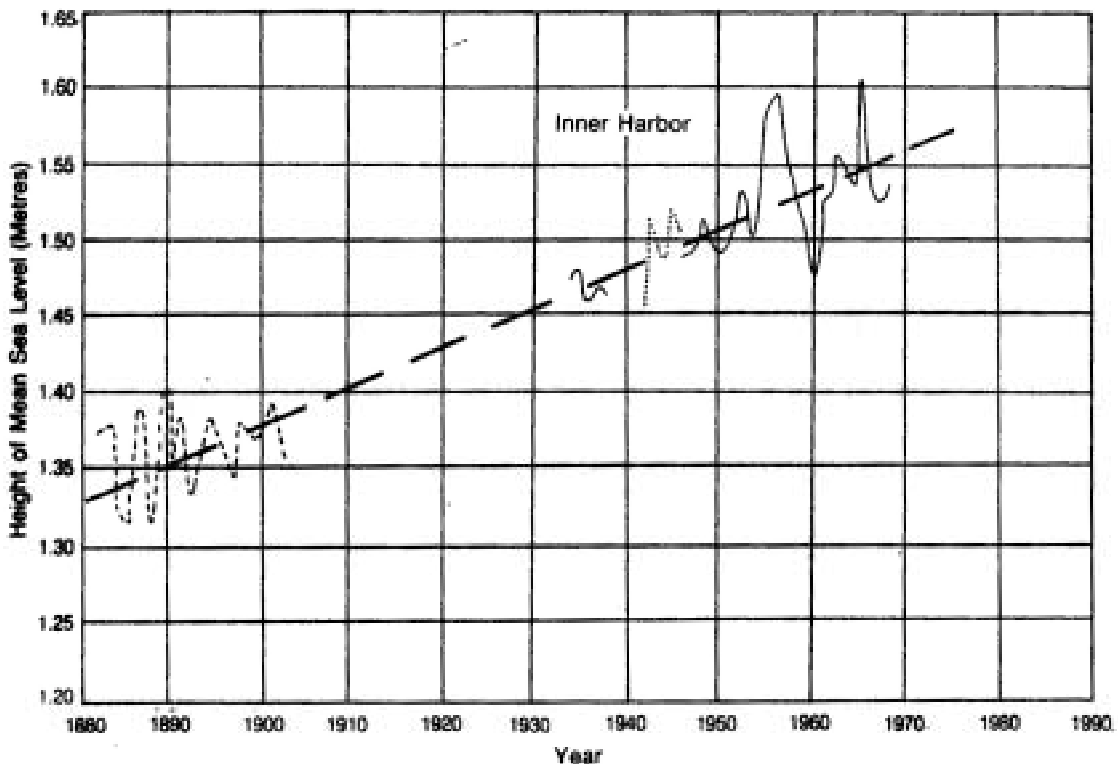


Figure 19A
From Culver Report

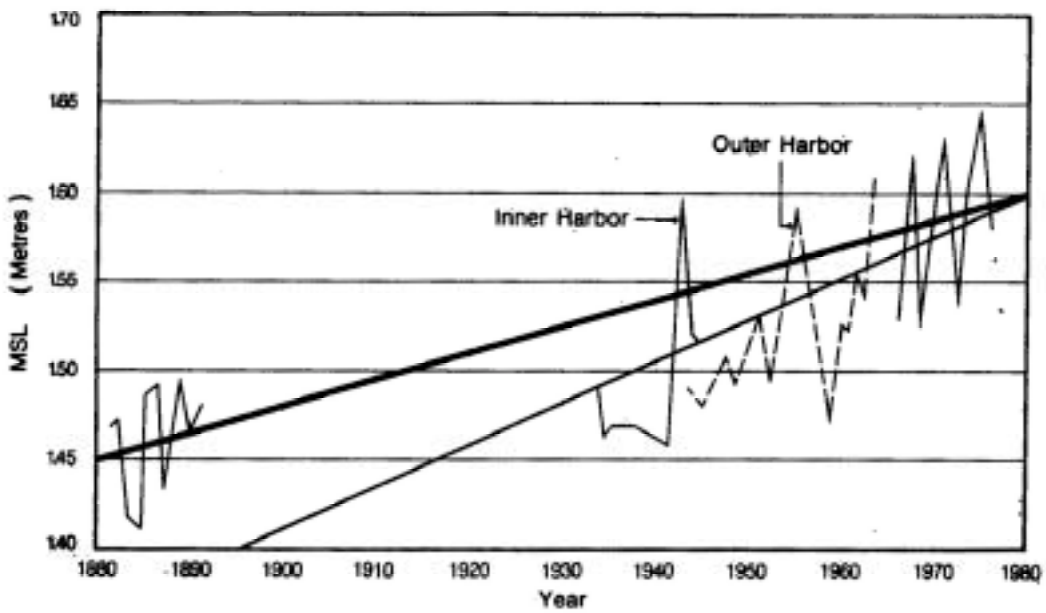


Figure 19B
DMH Values

Petrusevics (1982) examined seven Australian stations, applying linear regression analysis to the annual mean sea levels. Selected gap-free periods between 1966 and 1980 were used. The results are set out in Table 1.

TABLE 1 SEA LEVEL RISE AT AUSTRALIAN PORTS (PETRUSEVICS 1982)

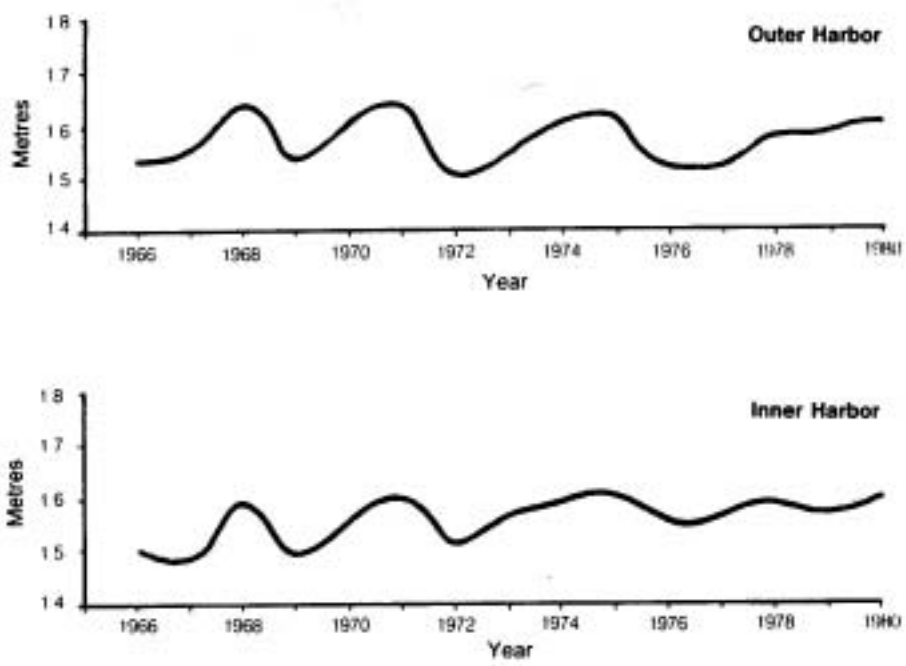
Station	Calculated Rise mm/year	r, confidence of fit coeff
Geraldton	2.8	0.28
Fremantle	0.75	0.10
Albany	0.77	0.10
Williamstown	7.8	0.70
Fort Denison	2.4	0.24
Townsville	1.9	0.28
Darwin	6.3	0.45

This shows the typical variation encountered when comparing changes at different places and provides limited information on the present trend of sea level rise. It is notable that the more reliable figures (those with a higher 'r' value) show significant rates of rise – between 2.4 and 7.8 mm a year.

Petrusevics also examined Inner and Outer Harbor data for the period between 1966 and 1980, and found consistent fluctuations of amplitude 50–70 mm and period 3 to 4 years. These are shown in Figure 20.

The foregoing information indicates that rapid short-term sea level rise, at rates of up to 7 or 8 mm a year, can be expected over periods of approximately 10 years, and also that shorter period fluctuations over a 3- to 4-year period are common in Gulf St Vincent.

FIGURE 20 ANNUAL MEAN SEA LEVEL INNER AND OUTER HARBOR, 1966–1980



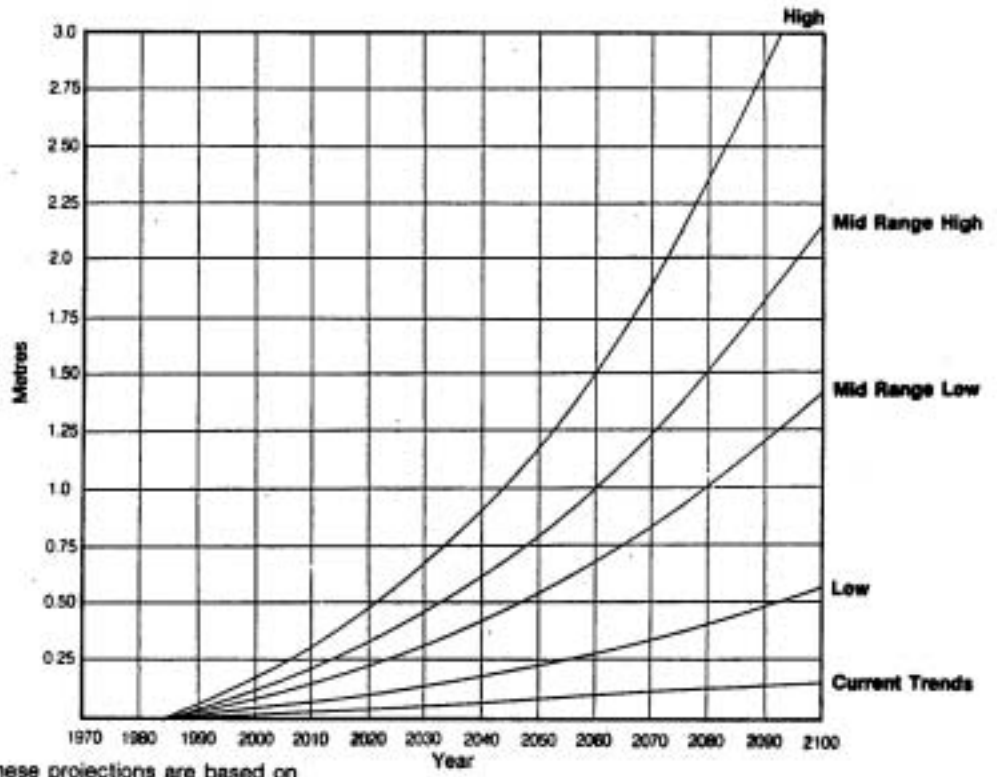
2.3.6 Future Sea Level Rise

There has been considerable scientific debate over recent years as to the likely effect of increased atmospheric carbon dioxide (mostly from power stations burning fossil fuels) causing warming of the earth by a 'greenhouse' effect. The effect of increased temperature would be widespread, and would include higher sea levels – through melting of polar ice and by expansion of the seawater. There is agreement on the rapid increase of $C0_2$, but trends in global temperature change are difficult to measure, because of annual fluctuations, and there has been less agreement on prediction of temperature rise. Both the temperature increase and the deduced sea level changes have been estimated using complex mathematical models, which have necessarily been run on estimated input data. The lack of verification has thrown doubt on these models and their predictions (Idso 1983). The question of sea level rise received considerable scientific attention in the USA in 1983, with a conference on sea level rise (held in Washington in March 1983) being followed by several scientific papers on the subject. These only became available towards the end of the present review, which takes account of their recommendations, insofar as weight can reasonably be given to them, and insofar as this was possible without major additional effort and delay.

The much publicised report, 'Can We Delay a Greenhouse Warming' (Siedel and Keyes 1983), and another, 'Projecting Future Sea Level Rise' (Hoffman et al 1983), were both released by the US Environmental Protection Agency late in 1983. These reports warn that the temperature rise will occur, and that ameliorative measures should be considered now. However, much research is still needed before reliable predictions can be made, and the rise may not be as extreme or as certain as press reports have suggested.

The EPA report considers that a global rise of between 1.44 m and 2.17 m by the year 2100 is most likely, but that the rise might be as low as 0.56 m or as high as 3.45 m. The rate would be accelerating rather than uniform. The EPA projections are shown in Figure 21.

FIGURE 21 FUTURE SEA LEVEL RISE – EPA PROJECTIONS (HOFFMAN ET AL 1983)



Note: These projections are based on unverified mathematical modelling—refer text.

The earlier part of the rise would be unlikely to be affected by possible collapse of ice sheets, which may eventually have a far greater effect. Collapse of the West Antarctic ice sheet, which is more vulnerable than the others because most of it is below water level, could lead to a rise in sea level of 5 to 6 m. The EPA 'sea level' report quotes studies that suggest that this could not happen in less than 200 years, and concludes that there is a possibility of complete disintegration of the ice sheet between 200 and 500 years. Similar collapse of ice sheets and rapid sea level rise is thought to have occurred during the past interglacial period.

The EPA's projection is that the rise over 50 years is most likely to be in the range 0.36 to 0.52 m, which is equivalent to an average rate of increase of 7 to 10 mm/year. However, much more validation by measurement of actual change is needed before these figures can be applied in an engineering design sense. The strategies have nevertheless been compared (in Section 5.10.7) for an average 10 mm/year rise over the 50-year study period. This is slightly above the EPA 'average' projection, and takes into account the possibility of some local land settlement.

2.4 TIDES, CURRENTS AND STORM SURGE

2.4.1 Astronomical Tides

Tides in the Southern Ocean and in the two gulfs are subject to the usual solar and lunar forces, and the response of the Southern Ocean to these is such that semi-diurnal tides occur (2 high waters and 2 low waters a day), and that there is a marked inequality between these two daily tides. The 'South Australian Sea' has resonance periods that influence the separate components of the diurnal tidal constituents in such a way that an apparently peculiar tidal behaviour occurs. The effects are different in each of the two gulfs. In the Gulf St Vincent, the entrance conditions create an apparent standing oscillation, which causes high tide to occur at the same time everywhere within the gulf. Both gulfs also experience an unusual situation known as 'dodge tides', which occur near the equinoxes and are due to tidal modifications causing water levels to remain constant for the whole day. The phenomenon, which also occurs to a lesser extent on other parts of the South Australian coast, is described by Bye in Twidale, Tyler and Webb (1976).

Tidal heights in the gulfs are larger than those along the adjacent ocean coasts. At Adelaide the variation in water level due to the astronomical tide is up to 2.5 m. This relatively large normal tidal range has a bearing on design of coast protection options.

2.4.2 Tidal and Other Currents

Currents are important to this study mainly insofar as they may be able to move sand into or out of the nearshore zone. They may also be important in determining the distribution and dispersion of pollutants that may be affecting seagrass growth. For both of these, currents associated with normal tides are probably more important than those with storm tides.

Space precludes including a full description of sea currents in the gulf. The reader is referred to Bye's description of the physical oceanography of the gulf in Twidale, Tyler and Webb (1976). This includes a description of the overall water circulation patterns.

No systematic investigation has been undertaken into currents off the study area, though various isolated projects provide some insight into the question. These are reviewed briefly here. Salinity and density studies have little bearing on the coastal processes, and work in this area is not included.

Bye (1974) modelled water circulation in the gulf (from a non-tidal point of view) and showed inflow along the west coast of the gulf, and outflow in the central region. His model suggested a northerly movement past the Adelaide metropolitan area.

Various current measurements at Port Stanvac in 1973 by Radok (1973) and Bye and Suter (1974) showed residual southerly movement during April (Radok) and residual northerly movement during

June to September (Bye and Suter). The latter movement was not confirmed by release of a free-floating buoy during the same period. The buoy indicated a residual southerly movement.

Radok (1977) used current meters for 3 months during 1977 to determine the current regime at the Port Adelaide sewage treatment outfall. He found that 'more often northerly flows were faster and lasted longer and generally northerly flows dominated at all distances from shore'. On the basis of this and his earlier work at Port Stanvac, he postulated a theory that there might be a circulation cell taking sand offshore from the northern end of the study area and returning it at some point to the south. There is little evidence to support this theory, which is not confirmed by bedforms or grain size analysis (See Section 2.2.4).

The Flinders Institute of Atmospheric and Marine Sciences measured currents off Hallett Cove for the Board in August and September 1979 (FIAMS 1979). A single current meter was used 4 m from the bottom in water of 10 m depth. A small net south-westerly drift of 5 cm/s was found. This was nearly parallel to the coast, though deviated slightly to the west (offshore). This drift occurred mainly in short bursts. Movement was in general dominated by the tides, though slightly less so than normally found in the gulf. Strong wind effects were noted on the currents, despite the 6 m depth.

The Coastal Management Branch has carried out various surface current investigations over the past 3 years. These include release of marked drift cards in 1980, radar tracking of offshore buoys in late 1980 and early 1981, and tracking of inshore buoys at Seacliff in early 1982.

In July 1980, drift cards were released 3 km, 6 km, and 12 km off Glenelg. Most of the cards were washed ashore to the south between Hallett Cove and Moana, the first being reported at 2 days, and the remainder within the next day or two. The prevailing wind direction during the experiment was from the north-west. Another release was made in mid October of the same year from a point 18 km to sea on a bearing of 262 degrees from the Holdfast Bay Sea Rescue Squadron Headquarters. Most of these cards came ashore along the metropolitan coast between Grange and Tennyson, after a period of 13 to 14 days at sea, having experienced a wide range of wind and tide conditions. These drift card experiments were an exploratory preliminary to the drogue buoy experiments, which were to follow, and provide little conclusive information.

Between September 1980 and April 1981, buoys fitted with radar transponders were tracked by radar, using equipment provided by the Sea Rescue Squadron (Petrusevics 1982). Four tracking experiments were done over a range of tide and wind conditions, with the buoys being placed at initial positions between 5.5 km and 18.5 km offshore from the Sea Rescue Squadron Headquarters. Buoys were tracked for periods of up to 3 days. This showed surface currents to be tidally dominated and aligned in a north-south direction approximately parallel to the coastline. Wind effects were found to be significant for winds exceeding 5 m/s. Maximum currents were found to be in the range 0.4 to 0.45 m/s (0.77 to 0.87 knots).

Nearshore current movements off Seacliff were investigated in March 1982 in conjunction with the blue sand tracer experiment (see Section 2.5.4) (Petrusevics 1983). Buoys were released between the breaker zone and a distance of 500 m offshore and were tracked using conventional survey equipment. The results of this work are still being assessed. A preliminary observation is that there appears to be a circulation cell in the area for some of the time during ebb tides. Northerly water movement was noted during some of the ebb tides, when water further offshore would have been moving in a southerly direction.

The possible effects of currents on the sediment budget for the study area are discussed in more depth under the heading 'Sand from Offshore' in Section 2.6.3.

Discussion here has been limited to normal tidal situations, as have most of the measurements described. Storm surge, discussed in the following section, would have considerably greater associated currents and, because of the storm waves associated with these events, would have a greater capacity to move bottom sediments. Little is known about these currents or their effects.

2.4.3 Barometric Pressure and Storm Surge

At Adelaide and elsewhere, significant coastal damage mainly occurs when large wind-generated sea waves occur during elevated tidal conditions. Little damage can occur when either extreme tides are accompanied by offshore winds (quite rare) or strong onshore winds (and consequential large waves) occur at low or medium tides. It is, however, unusual for storm waves not to be associated with some storm surge. The extent of this surge depends on the barometric pressure drop and, in Adelaide, on the strength and duration of winds from the north-west. These winds, which often occur in the early stages of a storm, cause the greatest surge effect. The higher water levels allow larger waves to reach the coast before breaking, and enable these waves to reach higher or further inland where they can do the most damage.

Extreme storm tide levels also cause damage and loss of life through flooding. Some flooding has occurred along the metropolitan coastline, mainly at Port Adelaide, during previous storms, including those in winter 1981. In the longer term, assuming a continually increasing mean sea level, this could become a major problem for Adelaide. However, it is not as pressing a problem as that of foreshore erosion, and is not addressed in this report.

The storm surge is the height by which the tide exceeds the predicted astronomical tide. Positive surges are invariably associated with and partly caused by low barometric pressure, and by wind patterns associated with travelling pressure systems or fronts. Winds blowing across an expanse of water will cause a water level rise known as wind set-up. This effect is exaggerated locally by the configuration of the gulfs in relation to the open ocean, and persistent winds from the west to north-west can cause a substantial ‘piling up’ of water in the gulfs. In addition to the hydrostatic and wind set-up level changes, other factors may apply, though the effect of these is not well known or predictable at Adelaide.

Computation of storm surge is complex and imprecise, especially for complex-shaped water bodies (as applies here). Because of this, and because of the lack of knowledge about some of the local factors, information from recorded extreme events is more reliable.

Additional factors that may contribute to the storm surge are natural oscillation of the semi-enclosed gulf waters and long-period ocean waves, both of which may be linked to moving weather systems. As will be discussed, the cumulative effect of all the factors is statistically remote and likely to be limited by other physical factors. Further discussion of the possible factors follows under separate headings.

Hydrostatic Water Level Rise

As atmospheric pressure drops, sea level rises to a hydrostatic balance. It is generally assumed that the change is in direct proportion to the relative specific gravities of water and mercury, ie under the old scale of barometric measurement a drop in pressure equivalent to 1 inch of mercury would result in a sea level rise of approximately 13 inches.

The generally accepted hydrostatic sea level rise (in SI units) is approximately 0.1 m for every 10 millibars. Local values in the gulfs may be higher because of dynamic amplifying factors. DMH obviously recognises some amplification in suggesting in its tide tables that 0.1 m of sea level rise be allowed for every 7 mB change in barometric pressure. Tronson and Noye (1973) examined the prediction of local storm surges by statistical analysis of 3 years of meteorological and tidal data. Their results showed that the Gulf St Vincent system might amplify the static response of sea level to air pressure by a factor of approximately 1.8.

Culver (1970) examined 30 years of barometric records and showed that the hourly average maximum and minimum barometric pressures were within + 1 inch Hg (33.9 mB) of the mean value in any month. This suggested that hydrostatic sea level rise was unlikely to exceed 0.5 m, assuming the higher DMH value. A greater pressure drop was recorded, however, during the storm of 3 July 1981.

Wind Set-up

In most instances, the effect of wind exceeds that of the pressure change. In Gulf St Vincent, this effect is greatest if winds persist in a north-westerly to westerly direction in the early stages of a storm. Such winds force water into the gulfs, significantly raising water levels.

The statistical analysis and prediction method developed by Tronson and Noye (1973) indicate a possible dynamic amplifying factor of up to 4 for the combined effect of air pressure and wind stress. This factor varies with the wind direction, being greatest for winds from the north-west. This 'dynamic amplification' may be one of several factors causing the surge levels to exceed normal expectation.

It seems unlikely that maximum hydrostatic sea level rise and maximum wind set-up can occur simultaneously. This is because the weather systems that cause the greatest pressure drop, the cyclonic low pressure systems, have rapidly changing winds near the centre of the low pressure, and these do not stay in one direction long enough to cause a large wind set-up. More distant or other weather systems generate more persistent winds, and hence greater wind set-up, but do not result in the same amount of pressure drop or hydrostatic water level rise.

Comparison of the storms of 1 June and 3 July 1981 is of interest. Both storms occurred at spring tide and both were associated with storm surges of approximately 1.4 m above the predicted tide.

On 1 June, the barometric pressure of 978 mB was the lowest ever recorded at Adelaide. The hydrostatic sea level rise associated with this was probably in the range 0.4 to 0.6 m. Wind set-up was calculated using a simplified method for enclosed waters (US Corps of Engineers 1973). The figure of 0.12 m obtained is likely to be less than would be obtained using a more rigorous approach and taking into account the gulf's interaction with the open ocean. It is not possible to determine to what extent the remaining surge should be attributed to an under-estimate of wind set-up or to the other factors mentioned.

On 3 July, the low pressure system was much further south, and the minimum pressure at Adelaide dropped to only 999 mB. However, winds were stronger and more persistent. The hydrostatic sea level rise would have been of the order of 0.2 to 0.3 m, whereas the wind set-up was calculated to be 0.46 m using the same method as for the 1 June event. The same uncertainty applies to the accuracy of the wind set-up calculation and the relative contribution of other factors.

Meteorological and tidal observations and surge calculations associated with these storms are described in Coastal Management Branch technical reports (Petrusevics 1981), and are illustrated in Figure 22.

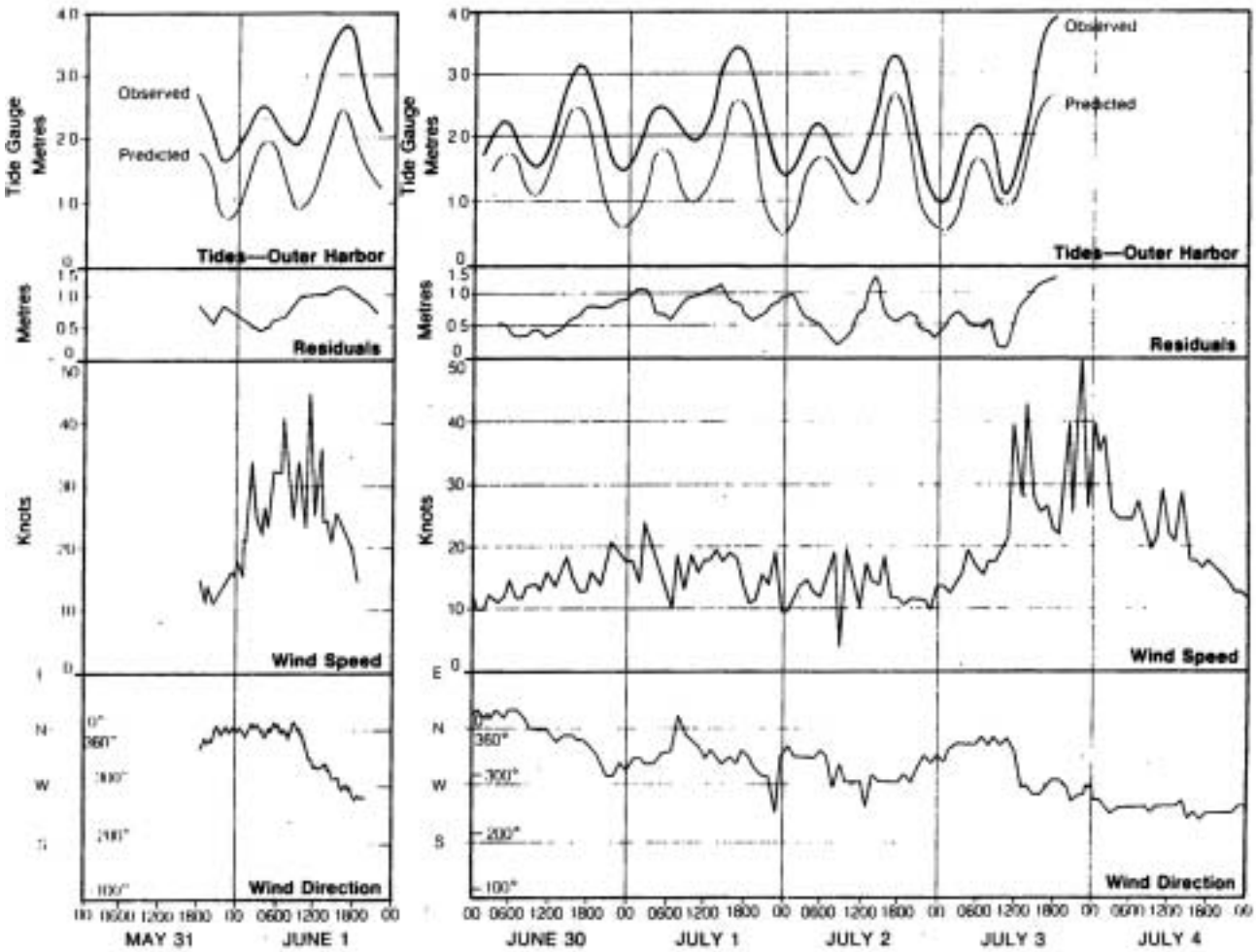
The maximum storm surge so far identified occurred on 2 December 1973 at low tide (Tonkin and Associates 1981). The water level was 1.6 m above the predicted tide. This surge was associated with only a small atmospheric pressure drop, and with persistent winds of 25 to 35 knots from the north-west. The winds and surge diminished before the following high tide, for which a surge of only 0.5 m applied. This event suggests that wind set-up may be the major component of storm surge at Adelaide. Data on the event is recorded in CMB Technical Report 83/6 (Moulds 1983).

Periodic Ocean Surges

Provis and Radok (1979) examined the occurrence of surges at tide gauges around the west, south and east coasts of Australia. They showed that there exist travelling surges that have travel times consistent with either the movement of the weather systems or with predicted periods of continental shelf waves. They consider that these surges are probably caused by weather patterns. Another explanation for these surges (Tonkin and Associates 1981) is that they may be caused by ocean eddies that travel eastwards from Western Australia with this frequency. These surges may cause a water rise of up to 0.5 m in gulf waters.

The occurrence of these surges appears to have now been well established, though their cause (or causes) still needs to be confirmed. It is likely that the presence of one of these travelling surges may at least partly account for the amount local tidal surges exceed the estimated meteorological ones.

FIGURE 22 STORM TIDE AND WIND RECORDS, 1 JUNE AND 3 JULY 1981



It is relevant to note that the maximum known surge, in December 1973, was preceded by an unexplained negative surge, and was not accompanied by extreme wind or atmospheric conditions, though was undoubtedly greatly influenced by strong north-westerly winds. The nature of the surge and the circumstances suggest the presence of a long-period wave.

Provis and Radok also noted the presence of another periodic water level fluctuation of longer period (between 20 and 365 days) which occurs virtually simultaneously at all stations on the Great Australian Bight.

Because of the apparent lack of east-west movement, they suggest that the source of these may lie in the Southern Ocean or in its interaction with the continent.

Natural Oscillation of Gulf St Vincent

Gulf St Vincent has a fundamental mode of oscillation calculated by Tronson (1975) to be 9.4 hours in a lengthwise direction. This is the period at which the gulf water would be likely to oscillate if set in motion by some extreme event such as an earthquake, a rapid change in meteorological conditions, or possibly an interaction with a very long-period ocean wave. Repeated occurrence of such events at or near the natural period of the gulf could result in large surges.

Tronson (1975) also calculated modes of oscillation for Spencer Gulf, and showed that there was a lack of interaction between the two gulfs in this regard, and that each could be examined separately.

No information is available on the likely magnitude of surges due to this cause, nor has its effect been confirmed and quantified.

Wave Set-up

It should be noted that the stillwater sea level inside the surf zone is slightly higher than that applying outside this zone or at the tide recorder locations. This is because breaking waves cause a build-up of water shoreward of the breaker zone. This build-up is dependent on the height of the breaking waves and on the beach slope, and cannot be predicted with great accuracy. Assuming deep water significant wave heights in the range of 2 to 3 m, and using the calculation methods set out in the US Corps of Engineers Beach Protection Manual 1973 and 1977, indicates wave set-up in the range 0.3 to 0.45 m. This will vary along the Adelaide coast, depending upon the beach slope and width of the breaker zone.

Wave set-up should not be confused with wave run-up, which is the height to which a particular wave will run up a certain slope. However, wave run-up computations usually include wave set-up. Calculation of wave run-up is not particularly useful for the Adelaide situation where seawalls are either rip-rap rock construction or older vertical concrete walls. Experience with existing seawalls has led to slight height modifications, and to a choice of top of the rock protection at AHD 4.2 m. Scouring that occurred in the fill material above some of the rip-rap walls built in the early 1970s was sufficient to warrant an increase in height from the earlier AHD 3.6 m.

Wave set-up needs to be taken into account in determining the depth of water against seawalls and the water levels inside the surf zone at places such as the Patawalonga and North Haven. The tide staff at the Patawalonga locks is influenced by wave set-up, and during storms it indicates higher tide levels than those recorded at the Outer Harbor gauge by approximately 0.2 m.

2.4.4 Maximum Storm Surge Levels and Return Periods

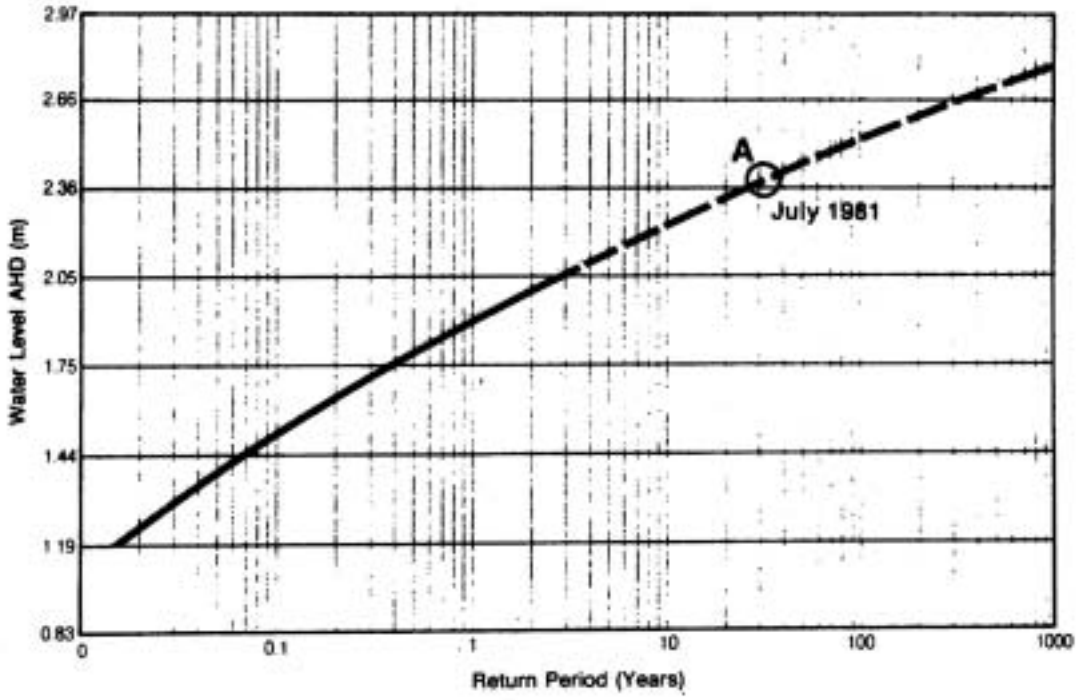
By examining a sufficiently long period of record, it is possible to estimate the likelihood, or return period, of a particular event occurring in the future. This is often necessary for engineering design – to balance expenditure against the avoidance of future damage. However, the limitations of this approach are increasingly being realised, and each situation really needs to be examined on its own merits, taking into account whether or not the event is at all acceptable, and its consequences.

A number of analyses have been made of extreme sea level occurrence at Adelaide (Pak Poy and Associates 1976; Tonkin and Associates 1981), and by DMH (Pickering, pers. comm., 1982). All their analyses pre-dated the record 1981 events, which significantly affect the evaluation.

The most useful analysis is that done by DMH using tidal records between 1915 and 1967. Figure 23 showing the frequency curve for high tides at Port Adelaide is based on DMH information. The solid line is based on analysis of records up to 1967, but does not include the earlier record tide of RL 114 feet (AHD 2.36 m), which was approached in 1972 (AHD 2.30 m recorded), or the two 1981 events, one of which equalled the 1960 record and one of which slightly exceeded this. The record 1981 tide is shown as point ‘A’ in Figure 23.

This suggests that the 1981 events should be regarded as of approximately 20 to 25 years return period, or possibly to have an even lower return period. The DMH analysis did not separate the effect that the ongoing sea level rise might be having, though this might account partly for the increased frequency of ‘record’ events.

FIGURE 23 EXTREME WATER LEVELS, PORT ADELAIDE (BASED ON DMH INFORMATION)



Further extrapolation of this curve, with some allowance for a further flattening with higher tide levels, suggests a 100-year extreme level of AHD 2.51, and an absolute extreme level of AHD 2.67. These latter figures are uncertain and are based on judgement rather than on probability prediction.

A proposal for re-analysis of the tidal records (including the more recent ones), and for application of recognised extreme value distribution functions to these, was considered. A statistically rigorous approach would have been costly and neither this nor a less rigorous approach considered would have been able to take into account limiting physical factors. It seems reasonable to expect that very large tides will be limited due to an increase in gulf surface area by overflowing and flooding of the extensive low-lying lands surrounding much of the gulf. An increasing restoring gravitational force would also inhibit extreme heights. It is interesting to note that the highest surge residual usually occurs at lower tides or while the tide is rising, but never at the maximum tide (see Figure 22). This suggests that inhibiting physical factors do apply.

The aforementioned maximum surge of 1.6 m, which occurred on a low tide, exceeds the surges recorded as part of the 1981 record tides. For the reasons previously given, it is likely that physical constraints either would have caused this surge to diminish with high tide or would have resulted in a lesser surge, had it occurred at high tide.

Discussion here has so far excluded the effect of an ongoing mean sea level rise (see Section 2.3), except insofar as it is incorporated within the tide records on which the predictions are based. Assuming a maximum rate of 2 mm a year for a 50-year design period, an additional amount of up to 0.1 m would be applicable, though would be mitigated by the factors discussed. Assuming its full application, the possible extreme tide levels are:

25-year return period	2.46 AHD
100-year return period	2.61 AHD
absolute extreme	2.77 AHD

Higher levels would occur if the projected 'greenhouse effect' sea level rise eventuates (see Section 2.3.6).

Stillwater levels within the surf zone would be 0.2 to 0.3 m higher, due to wave set-up.

As will be discussed later in this report, these extreme water level predictions, while important in considering flooding of low-lying areas, such as at Port Adelaide, do not have a great bearing on the comparison of alternatives for protection of the coast between Kingston Park and Outer Harbor over the chosen design period.

2.4.5 Storm Surge and Rainfall Occurrence

Storm surge can result in drain outlets being 'drowned' and in stormwater being backed up. Co-incident occurrence of storm surge and heavy rainfall would be likely to result in severe flooding. Fortunately, there appears to be only a weak correlation between these two types of events, possibly because they are caused by different weather conditions.

Statistical analyses of rainfall and storm tide occurrences were made by Beales (1971) and by Tonkin and Associates (1981). Beales examined the 90 largest rainstorms between 1912 and 1961 and found the mean tide during these events to be an average of 0.5 feet (approximately 0.15 m) higher than normal mean sea level. He found a slight negative correlation between sea level height and rainfall, with a slight tendency for sea level to be lower than normal during extreme rainfall events.

Tonkin and Associates examined the correlation between storm surge and rainfall for the period 1966–78, which included 80 storms of duration between 3 hours and 24 hours. They found no significant dependence between the amount of rainfall and the amount of surge, though a statistically significant difference was found between the mean of the surges coincident with rainfall events and the mean of all surges. Like Beales, they concluded that surges associated with extreme rainfall events are statistically lower than those not associated with these events. Tonkin and Associates concluded that, for storms of 6 hours duration or more, a typical associated positive surge was 0.2 m, and they recommended that this surge be adopted for all rainfall recurrence intervals in the area for which the study was done.

2.4.6 Storm Predictability

It would be useful, for the purposes of constructing coastal protection works or undertaking major beach replenishment, to have some indication as to whether any particular year or sequence of years would be likely to have a higher than normal chance of being stormy or of providing a major storm. However, as with other aspects of long-term climatic prediction, this remains an unattainable ideal. There has been a persistent local 'truism' that major storms affect the Adelaide coast at approximately 20-year intervals. This is not borne out of the examination of storm occurrence. Major storms occurred in 1948, 1953, 1972 and 1981. These dates are based on a Coastal Management Branch collation of information on all significant local coastal storms (Petrusevics 1981). Reliance was necessarily placed on newspaper reports and other incomplete records, and the limitations of storm comparison are obvious. Not only is the measure of the intensity a function of tide level, wind intensity and its duration from a particular direction, but storm damage, as reported for the events, is also dependent on the standard of coast protection provided, and on the pre-storm condition of the beach.

There is a tendency for storms to occur in groups, as with other meteorological events such as wet summers, hot spells, and so on. Combinations of upper atmospheric conditions and high sea surface temperatures, favourable to the generation of stormy weather systems, can persist for 6 to 12 months (Kemp and Douglas 1981). A severe storm is more likely than not to be followed by another within a month or two.

Examination of damaging storms on the Adelaide coast shows a high preference for certain months of the year. Most significant storms occur during May and June and approximately half as many during July and August. A few major storms have occurred in spring and early summer, but these events are rare. January, February and March are the calmest months, with no major storms having been recorded. These are obviously the best months for carrying out coastal works.

2.5 WIND, WAVES AND ALONGSHORE SEDIMENT TRANSPORT

2.5.1 Introduction

The Adelaide coastal erosion problem is mainly due to the inadequate amount of sand on the beaches and to its predominant movement from south to north. A knowledge of this sand movement is essential to an understanding of the erosion problems and is critical to the design and comparison of alternative coast protection methods. The sand movement cannot be fully understood or estimated without a knowledge of the winds and waves that cause its movement.

Sand is mainly moved along a coast by an alongshore current, which is induced by waves striking the coast at an angle. The turbulence created by the waves assists in this process, which occurs across the active beach zone.

The configuration of the water body in relation to the land masses is important because it affects the ability of winds from various directions to create waves affecting the study area. Both the height and period of waves are related to the fetch (the distance over which the wind can generate waves by action on the sea surface). Figure 24A shows the gulf configuration and Figure 24B shows the fetch/direction relationship for different parts of the study coastline.

A knowledge of the strength and occurrence of winds from offshore directions is critical, as these generate the waves. This data must be sufficient to enable the calculation of growth and decay of waves as the wind changes strength and direction. This usually means that hourly wind data must be available.

On most coastlines, ocean swell generated by distant winds makes up a significant part of the total wave energy. However, for the study area the effect of swell is unimportant in relation to local wind-generated waves (Culver 1970). This is because ocean swell is heavily attenuated by the narrow entrances to the gulf and by its shallowness.

Bathometry must be known, as water depth influences both the generation of waves and their refraction as they approach the shore. The effect of bottom friction is such that wave generation in 'shallow' water, as applies for the gulfs, is different to that for 'deep' water, and the prediction method must take account of water depth.

Wave refraction is the bending of wave crests as waves enter shallow water and the crests tend towards a closer alignment with the bottom contours. The effect depends on the relation of water depth to wavelength, and is analogous to refraction for other types of waves such as light and sound waves. It is due to that part of a wave in deeper water moving faster than the part in shallower water, because of the relationship between wave celerity and water depth. Refraction is important in that it is the main factor influencing the distribution of wave energy along a coastline. It explains the focussing of wave energy on headlands, the dispersion in bays, and other coastal energy concentrations due to offshore topography. Coastal wave breaking and wave energy direction is influenced by refraction, which therefore needs to be taken into account in sediment transport calculation.

Wave reflection is also important as this can result in additional turbulence and scouring at coastal structures. It can also cause high instantaneous water levels due to superposition of incident and reflected waves.

Wave diffraction is another phenomenon that sometimes needs to be taken into account in the estimation of nearshore wave direction and energy, but it is not of great importance in the study area. Diffraction is the bending of wave crests when a train of waves is interrupted by a barrier of some sort. The diffracted waves enter the otherwise sheltered area behind the obstacle by a process of lateral energy transfer along the wave crests. Where diffraction is significant, coastal waves are determined by a combined refraction/diffraction analysis process. Diffraction has not been taken into account in any of the studies for the Adelaide coast, though may apply to a slight extent at the Kingston Park end and at structures.

FIGURE 24 WAVE GENERATION

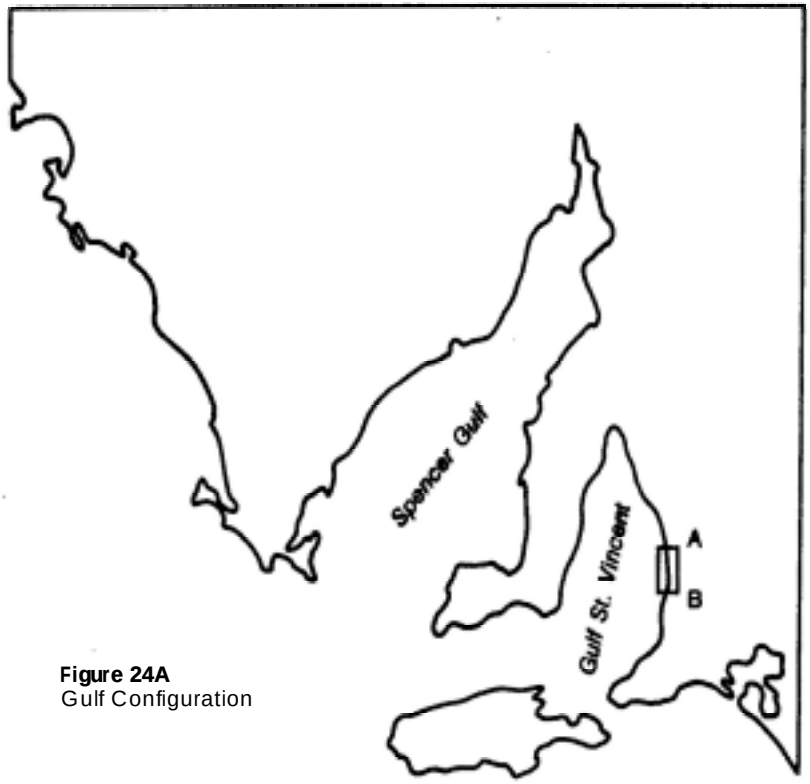


Figure 24A
Gulf Configuration

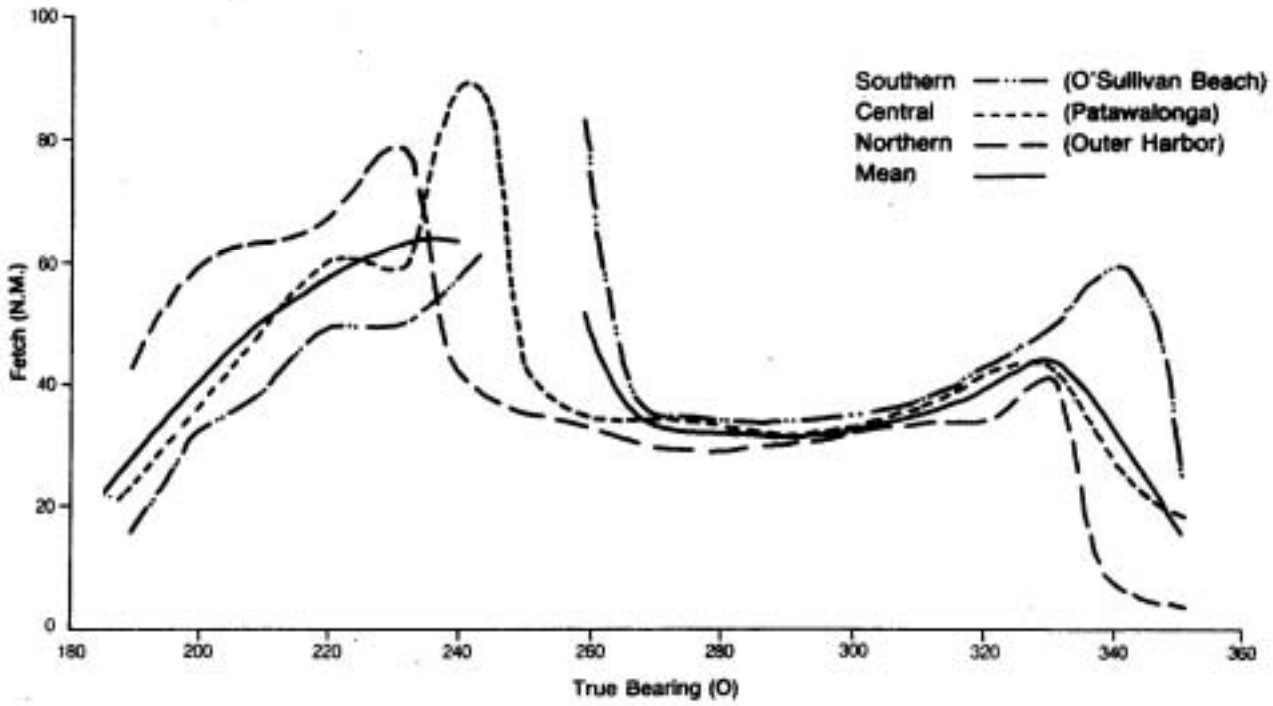


Figure 24B
Fetch/Direction Relationship (Culver 1970)

Wave data is either obtained using wave recording devices or by calculation using empirical or semi-empirical/theoretical methods. Wave recorders are expensive to maintain, and a common practice, which has been adopted locally, is to obtain sufficient wave records to enable the development and proving of a reliable wave prediction model. Waves are then 'hindcast' or predicted using wind data.

Ideally information is required on sand movement in both alongshore directions, and also onshore and offshore, for all parts of the study area throughout the year. Likely variations from year to year should also desirably be known. In practice, such knowledge is only partially attainable, and engineering solutions must be based on much less information. Deduction based on the effect of existing structures on sand movement is consequently invaluable, both to validate the sand transport calculations, and to provide supplementary, more reliable information.

Empirical relationships, based on a large amount of observation, have been derived between alongshore sediment movement and the alongshore component of wave energy. These are, however, capable of giving only the order of transport and contain some very limiting assumptions. Nevertheless, if confirmed or adjusted according to known information such as measured accumulations at beach structures, they can provide information essential to the design of coast protection strategies.

The usual procedure is thus to obtain offshore wave data, either using a wave recorder or, by calculation based on wind data, to calculate the nearshore wave refraction using local bathometry and hence to estimate the alongshore component of wave energy – and then to calculate the alongshore sand movement in both directions, as may apply. The gross movement or the net movement in a predominant direction is then checked against measured sediment erosion and accumulation quantities if these are available. Experiments using an identifiable tracer sand may provide useful supplementary information, but cannot provide reliable transport rates.

2.5.2 Winds

A most significant characteristic of Adelaide wind is its variability, both during a particular year, and from year to year. This variation results in significant fluctuation in sand transport direction during a year and in the total amount of sand moved from year to year.

Typical Wind Distribution

A typical/average annual wind rose for Adelaide airport is shown in Figure 25. This shows very clearly that the most significant wind activity is from the south-west quarter, though it also shows a substantial amount from the north-east quarter. Being off the land, the latter has little effect on coastal processes. The rose also shows that significant activity occurs from the north-west quarter.

The seasonal wind distribution shown in Figure 25 is derived from 3-hourly records provided by the Bureau of Meteorology. These wind roses confirm previous observations (Culver 1970) based on a more detailed analysis of a shorter data set at closer time intervals. Culver's description of the wind pattern is as follows:

> There appears to be a relatively regular pattern evidenced each year. In January, February and March, the predominant wind pattern is primarily south-west with some south-east and north-east wind. Late March and April shows a well distributed wind pattern (right round the compass). In May, north-west and south-west winds begin to show longer durations and higher velocities. This pattern continues into June and July often with stronger north-west winds predominating (with some north-east activity). West to south-west winds begin to blow stronger in August and continue into September with increasing velocities. The equinox is traditionally squally and the September–October period shows strong winds between the north-west and south-west quarters. The distribution in November is again well round the card but with longer duration west-south-west winds evident. In December, wind velocities are lower with south-west winds predominating with some north-east activity closely paralleling the January and February pattern.

FIGURE 25 WIND ROSES, ADELAIDE AIRPORT

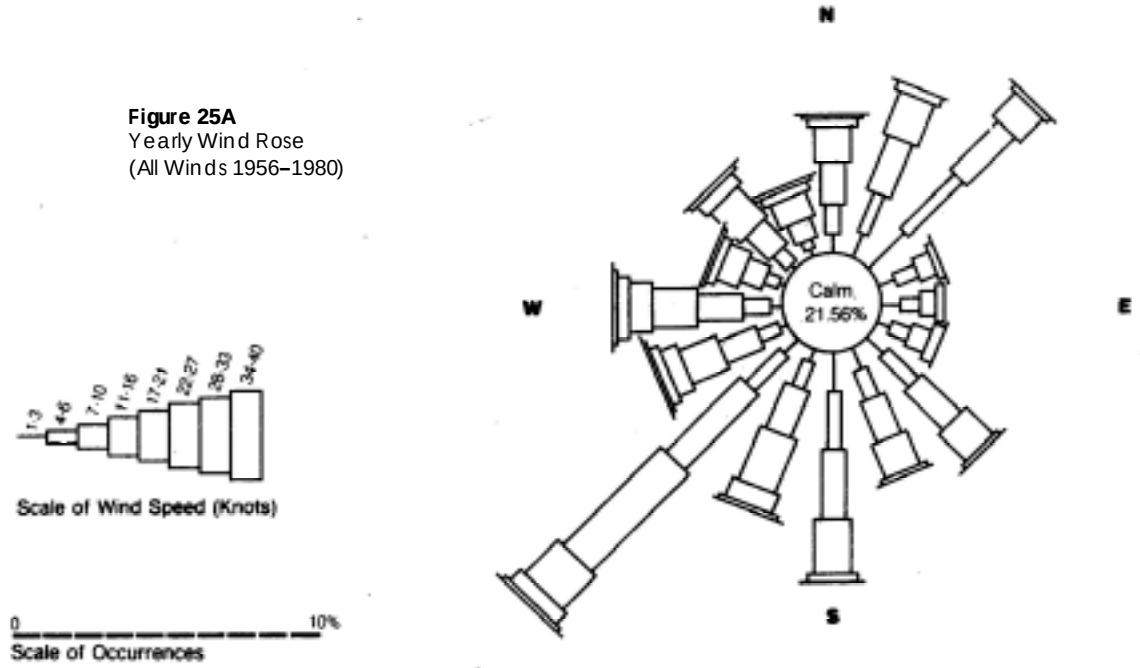


Figure 25A
Yearly Wind Rose
(All Winds 1956–1980)

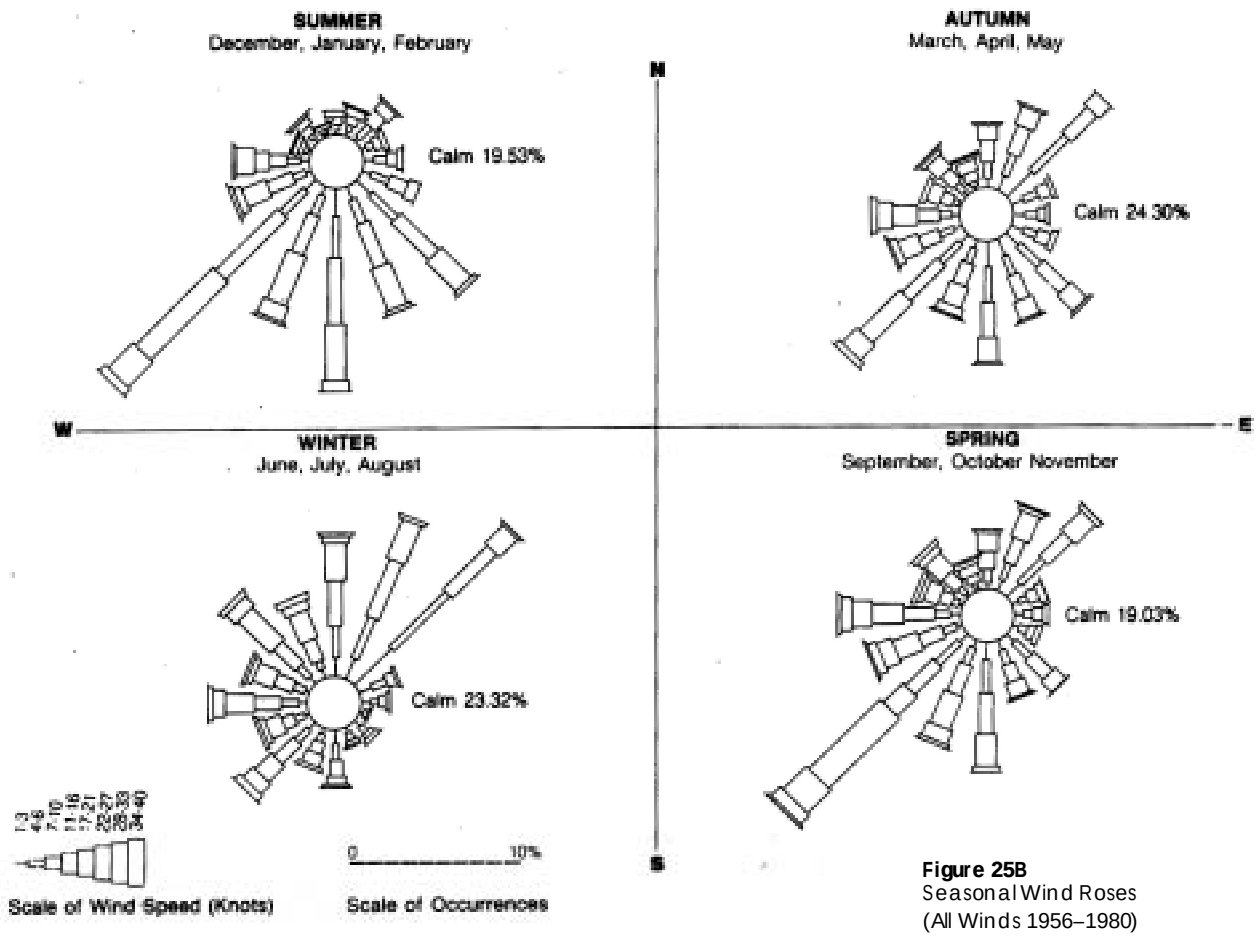


Figure 25B
Seasonal Wind Roses
(All Winds 1956–1980)

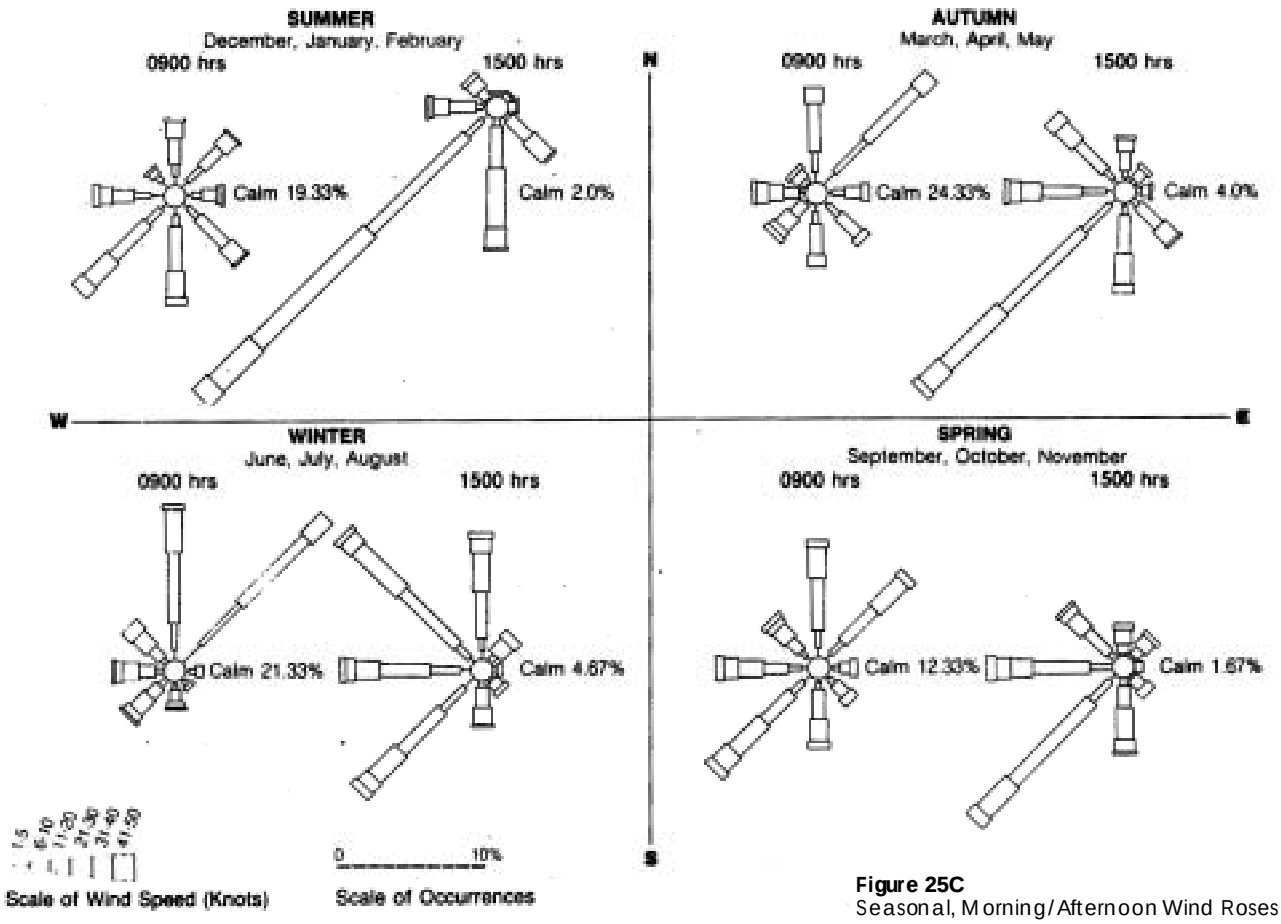


Figure 25C
Seasonal, Morning/Afternoon Wind Roses

There is considerable wind variation with time of day as shown in Figure 25C. The figure clearly illustrates the contribution of sea breezes and the seasonal distribution of this.

Both the daily and seasonal wind distribution influence the economics of operating a dredging plant for beach replenishment or for dredging the Patawalonga channel. Section 2.5.4 includes discussion of the seasonal and shorter-term wind variations on alongshore sand movement.

Wind Analysis

Wind analysis was done by Culver (1970) and, to a lesser degree, as part of the present study. Culver digitised 6 years of hourly data from the previous Bureau of Meteorology recording station on West Terrace, and used this data in his analysis. He collated the data into wind roses and total wind run, and did a quite detailed analysis to determine the occurrence and duration of winds of various intensities for each month of the year. Wind analysis in the present study used 3-hourly records from the Adelaide Airport, provided by the Bureau of Meteorology.

These 3-hourly records provide enough information for useful wind analysis, but are not generally considered sufficient for reliable wave prediction. This is because wave generation is sensitive to changes in wind strength and direction, and the duration of a particular wave climate may be much less than 3 hours. The scope of the present study did not allow for digitising lengthy wind data sets, as was thought to be necessary for a more detailed analysis of winds or for hindcasting of waves. However, half-hourly data is available for future use. The information, which is available on the daily weather record sheets at the Adelaide Airport, could be copied and transferred to computer tape. This would cost approximately $5,000 for 10 years of data. Culver and Walker (1983) report that they tested the 3-hourly wind data for long-term accumulated wave energy prediction (refer to Section 2.5.3), and that it proved satisfactory for this purpose.

The 3-hourly records were analysed for 18 years of data – the years 1969 to 1981 inclusive, and the years 1957, 1958, 1959, 1960 and 1964, to correspond with the earlier work by Culver. The computer analysis for this was done by the University Civil Engineering Department.

For each year, the resultant wind run was obtained for all winds, and separately for onshore winds only. Resultants were also computed for both these cases, applying thresholds of 13 and 17 knots. These correspond to the 15 and 20 mph thresholds applied by Culver, and were chosen, following Culver, as being the most useful indication of that component of the wind having most influence on coastal processes. Figure 26A shows the annual resultant wind runs for each year, and Figure 26B shows the alongshore component of these.

Annual Wind Variation

The predominance of winds in a west-south-west to south-west direction (with consequent northerly, alongshore component) is evident from these, as is a fair amount of variation in both total wind for each year and in the resultant direction.

Comparison with Culver's observations is of interest. On the records that he analysed, Culver noted that both 1956 and 1964 were unusual years, the former because the wind from the north-west quarter exceeded that from the south-west, and the latter because of the small northerly component and an apparent anomalous beach behaviour. Unfortunately, the records obtained from the Bureau of Meteorology did not include 1956, and comparison cannot be made for that year. However, 1981, which was unusually stormy, appears to have been similar in having a very small wind component in a northerly direction. In 1981 the alongshore wind component of onshore winds over 17 knots was in a southerly direction. It is noteworthy that, for the period examined, this was the only year in which this occurred (see Figure 26B).

It is obvious from examination of Figure 26A that 1964 was not as atypical a year as Culver's examination of a shorter period of record had suggested.

Wind Resultant and Annual Alongshore Drift

Both the annual wind vector and the magnitude of the alongshore component give useful clues as to which years could be expected to have had large or small alongshore sand drift quantities. They also show that the quantity could be expected to be highly variable from year to year. It must, however, be remembered that alongshore sand movement is related to the amount and direction of wave energy, and not to the wind. The combined effects of varying fetch and of refraction result in local wave energy having a more northerly resultant direction than does the onshore wind. Variation in the annual direction of the resultant is also less. Culver (1970) showed that a northerly annual sand movement will occur even though the onshore wind resultant may have a southerly component.

In addition to being influenced by water depth and fetch, wave generation depends on the duration of winds, consistency in strength and direction, and wave height is related to the square of the wind velocity. The annual variation in wave energy may therefore be quite different to that of the onshore wind resultant. Undue reliance should not be placed on wind analysis in deducing alongshore drift.

Notwithstanding this, an attempt was made to deduce an approximate relationship between annual alongshore sand movement and the annual wind run resultant using the calculated alongshore sand drift quantities from the Culver report (Moulds 1982). This was unsuccessful because of the few points available and the scatter of these. By using shorter time periods, it may be possible to derive a local empirical relationship between wind and alongshore sand movement. However, the limitations of this approach are such that it should be seen as providing a useful indicator, rather than being a substitute for calculating long-term wave energy and obtaining values of alongshore sand drift from this.

FIGURE 26 ANNUAL ONSHORE WIND RESULTANTS

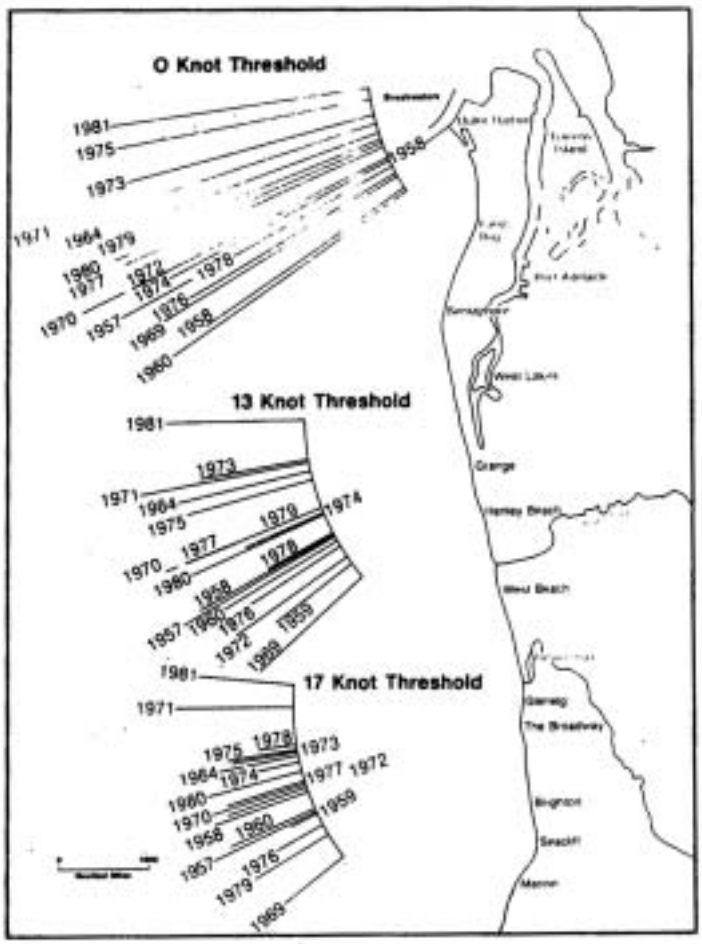


Figure 26A
Resultant Vectors

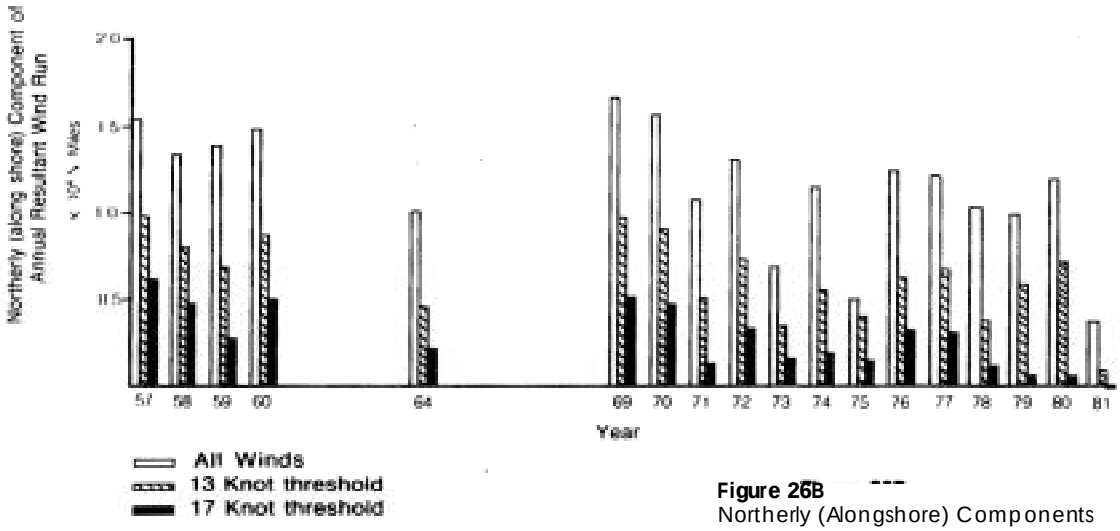


Figure 26B
Northerly (Alongshore) Components

Wind-Blown Sand

Culver (1970) showed that the effect that wind has on moving dry beach sand might be significant in the local situation.

The alongshore movement of wind-blown sand is limited by the width of dry sand on the beach. Assuming Culver's calculated average annual net northerly movement of 550 lb/year/feet of dry sand beach width, simple calculation shows that the annual northwards movement is less than 20 m^3 of sand, for even the widest beaches. This is very small in relation to the estimated 30,000 m^3 moved northwards each year by the waves, and can obviously be ignored.

Potential for inland movement is greater because of the greater wind component in this direction and because of the greater width over which movement can occur. Culver suggests that large losses would probably have occurred in the past when the coastal dunes were less stable than they are now – particularly during the earlier part of this century and during early development of the coast for housing. He shows that wind deflation could have caused beach loss at the rate of 30 feet (approximately 9 m) a century at places along the Adelaide coast.

Windblown loss of sand from the Adelaide beaches has now virtually ceased except at the West Beach sand dunes. Planting of sand dunes and the construction of sand-drift fencing has almost completely stopped drift at most places, and the small amount of sand that does reach esplanade roads is returned to the beach by local councils. The considerable length of rock seawall constructed in recent years has also reduced beach width (resulting in less dry sand on the beaches) and has provided a physical barrier against drift.

Deflation loss associated with a possible major replenishment of the southern beaches would present an unacceptable nuisance to the public before it resulted in a serious loss of sand from the beach. Assuming no restriction to sand drift (which obviously would not apply), and assuming Culver's calculated average annual rate of 3,020 lb of drift through each north-south foot of beach, gives a hypothetical total inland drift of approximately 20,000 m^3 a year for a 7 km length of replenished beach. While this rate is unrealistically high, it serves to emphasise the importance of sand drift control, particularly if a major beach replenishment were to be undertaken.

Wind has contributed significantly to inland movement of the dunes at the West Beach Recreation Reserve, and to consequent erosion of the front face of these dunes (Beare and Moulds 1981). This has been associated with loss of vegetation from the dunes and has slowed down in recent years due to planting arranged by the Branch. The situation at West Beach is discussed in more detail in sections 2.6.2 and 2.6.3.

2.5.3 Waves

Both maximum storm waves and longer-term average wave climate and energy needs to be known – the former for the design of coastal structures to resist wave forces, and the latter to understand and predict sand movement along the beach.

The Wave Study

In 1980, the Board provided a grant to the University of Adelaide for a study of waves in Gulf St Vincent, and for the development of a system of remote data gathering. This work was jointly sponsored by DMH, which provided services such as the deployment of wave recorders and their maintenance.

The study included the refining of an existing system of intelligent computer data collection from a recorder close to Adelaide, using VHF radio communication. This system was used for a wave recorder that was moved to a station off Seacliff for this study. The University had been involved with DMH in initial development of the remote system while the wave recorder was at a previous station off Outer Harbor. The Seacliff data has been collected and analysed for 2 years, and has been compiled into a 'wave atlas' (Culver and Walker 1983). Data continues to be collected from this recorder.

A related developmental aspect of the study was the establishment of a system for the computer in the Civil Engineering Department to automatically collect data from recorders beyond VHF radio range. This system, which is now operational, was established for two DMH recorders in the Upper Spencer Gulf at Wallaroo and Tickera. Data is temporarily stored at a land station within VHF radio distance of the recorders and is collected at 'predetermined intervals by the computer using a standard STD telephone line. The method is very economical and reliable, and will also be applicable for other, future remote use.

The study included the development of a sophisticated wave prediction model for Gulf St Vincent, and an alternative version of this, which gives slightly less wave information, but which operates more quickly and is better suited for use in calculating sand movement along the Adelaide coastline.

Although not initially envisaged as a major part of the study, a compartmentalised alongshore sediment transport model was devised for the Adelaide study area. This shows much promise despite being at a preliminary stage.

This description is necessarily brief as only a part of the work has a direct bearing on the present review. Relevant aspects are discussed in more detail where relevant to other sections of this report. Full details of the study are available in a series of reports produced on the work (Culver and Walker 1983).

Wave Measurement

Wave measurement is usually limited to the minimum period necessary to provide a statistically reliable picture of the wave climate and extreme events. There is considerable difference of opinion about the length of record required. A 2-year recording period is usual for engineering design purposes. Although this may not give entirely reliable information on extreme events, it is more than sufficient for the development and testing of wave prediction models.

The most useful wave records for the study area are those obtained from the Seacliff recorder. The recorder's location is shown in Figure 27A. Other wave recording of some relevance includes that undertaken at the Grange jetty during the last 2 years of the Culver study (1968–70). Records from the Seacliff recorder's previous location off Outer Harbor are less relevant and contain large gaps.

The Seacliff record, from January 1981 until the present, has included both the unusually stormy year 1981 and the very calm 1982. It has continued through 1983, this being a more typical year.

The digital wave recorder at Seacliff is one of three built and owned by DMH. Its layout and operation is shown diagrammatically in Figure 27B. The instrument is mounted on a large tripod, which stands in approximately 10 m of water. It is a float type surface sensing device. A polystyrene float containing magnets floats freely on the outside of a tube containing magnetic sensitive relay switches, and activates the relays as it moves up and down. The float and tube are contained within a protective chamber.

The relays generate a simple binary code, which is transmitted by a solar powered VHF radio to a shore station at the University. The signal is fed directly into a DEC PDP – 11/34 computer, which controls the frequency of signalling and recording, as well as screening and processing the data. The system was developed and run by the Civil Engineering Department, University of Adelaide, as part of a study commissioned jointly by the Coast Protection Board and DMH. It is described more fully in Walker (1980) and Culver and Walker (1983).

The procedure has been to collect data for 10 minutes every 2 hours under calm to moderate conditions, and to collect a 35-minute record every 6 hours. The first 10 minutes of data is edited by the computer and significant wave height calculated. The sampling frequency is changed to every hour when the significant wave height exceeds one metre or when the recorded wind velocity is greater than 25 m/s. The results of the significant wave height and spectral analysis, which are done on all records, are stored on magnetic tape, as is the raw data from the 35-minute records.

FIGURE 27 THE SEACLIFF WAVE RECORDER (CULVER AND WALKER 1983)

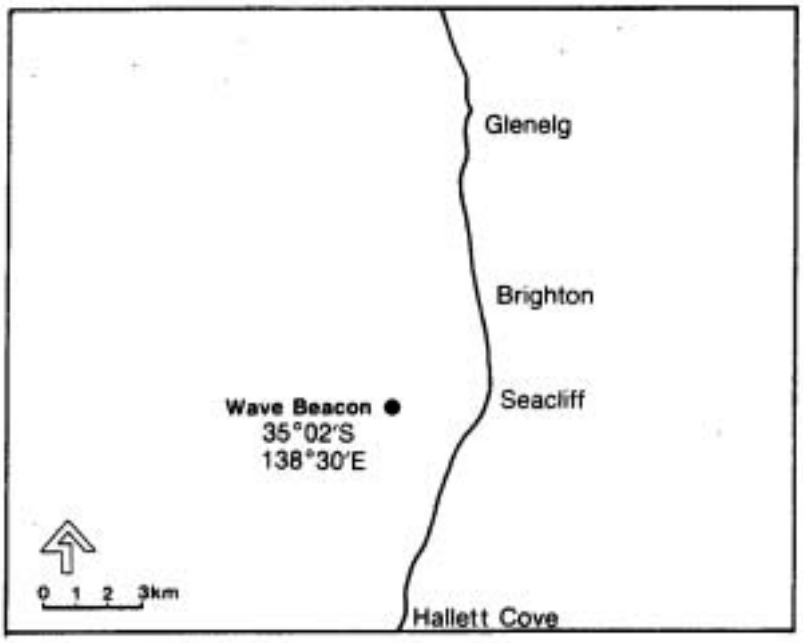


Figure 27A
Location

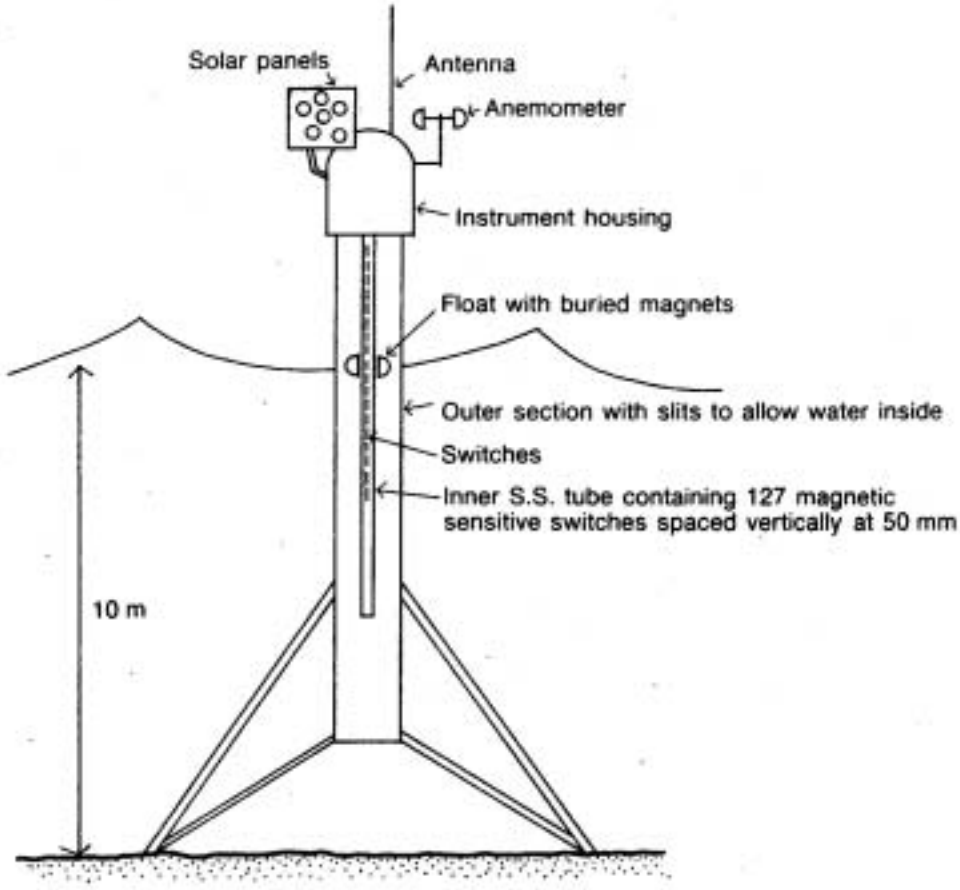


Figure 27B
Arrangement and Operation

The wave recorder has proved extremely reliable in comparison with other wave recording systems in general use, only two breaks in transmission having occurred during the 2-year period of operation. There are other minor gaps in the wave record associated with faults at the receiver/computer end.

An example of information from the Seacliff wave recorder is provided in figures 28 and 29. Further analysis, including monthly wave height exceedence and persistence plots, is included in the 'Seacliff Wave Atlas' (Culver and Walker 1983).

The University's review of the use of satellite data for sea state assessment suggests that the 'footprint size' is such that wavelength cannot be assessed, but that significant wave height can be deduced. The University has examined data from the satellites GEOS-3 and SEASAT-A. The Wave Study included the design of a system to enable wave direction to be recorded at the Seacliff beacon. Unfortunately, it proved impossible to implement this within the study timetable. The proposed system was to rely on an array of four bottom pressure gauges with appropriate extra electronics to enable the additional information to be transmitted via the radio link to the University.

Wave Prediction

Two wave prediction systems for waves in intermediate water depths have been modified and applied locally by the University of Adelaide Civil Engineering Department. The first, using a variation of the SMB (Sverdrup, Muuk, Bretsneider) method, was used in and formed the basis of much of the Culver Report. The second, more theoretical and powerful, method has been developed and refined as part of a study partly funded by the Board.

The first method warrants a brief description, although it has not been used in the present review and has been largely superseded by the more recent work. It has, however, formed the basis of the only long-term wave prediction and energy calculation done for the Adelaide coast, and underlies our present understanding of the alongshore sediment movement.

a) SMB Method Used in the Culver Report

The main wave prediction method used in the 1970 study was to solve the Bretsneider wave equations along the fetch length using modifications of a procedure suggested by Ijima and Tang (1966). The method is described in Culver 1970. The equations that are solved incrementally along the fetch length are:

$$\frac{gH}{u^2} = A \tanh K3 \left(\frac{gD}{u^2}\right)^{\frac{3}{4}} . \tanh \frac{K1 \left(\frac{gx}{u^2}\right)}{\tanh K3 \left(\frac{gD}{u^2}\right)^{\frac{3}{4}}}$$

and:

$$\frac{gT}{2\pi u} = B \tanh K4 \left(\frac{gD}{u^2}\right)^{\frac{3}{8}} . \tanh \frac{K2 \left(\frac{gx}{u^2}\right)^{\frac{1}{3}}}{\tanh K4 \left(\frac{gD}{u^2}\right)^{\frac{3}{8}}}$$

where H = Significant Wave Height (ft)
T = Significant Wave Period (sec)
g = gravitational acceleration
D = water depth
x = Fetch Length
A, B, K, K_2, K_3, K_4 are fitted constants.

FIGURE 28 SEACLIFF WAVE DATA, 1981 AND 1982 (CULVER AND WALKER 1983)

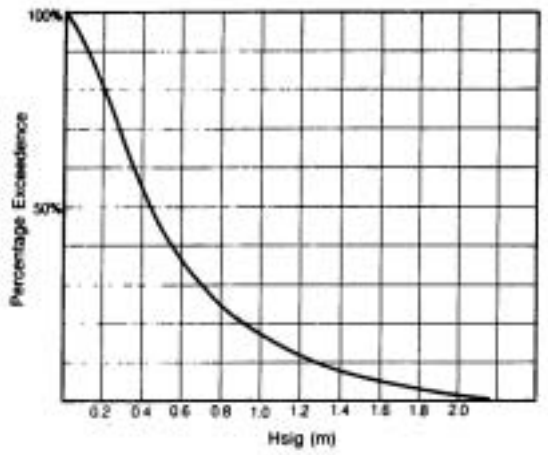


Figure 28A
HSig, Percentage Exceedence

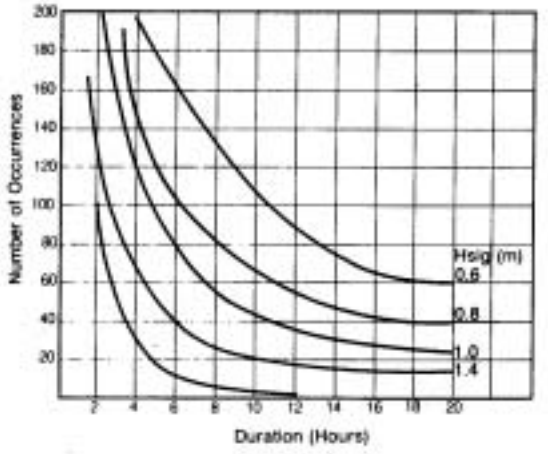


Figure 28B
Hsig, Duration, Number of Occurrences/Year

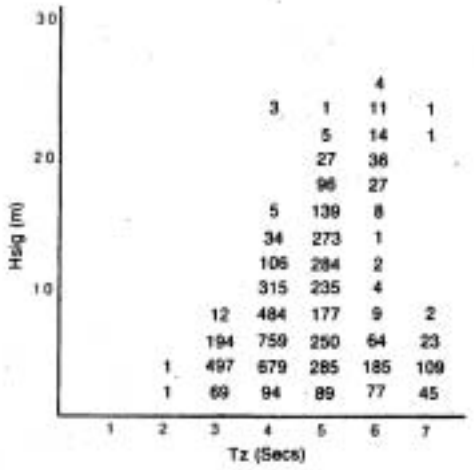


Figure 28C
Hsig–Tz Scatter Diagram

FIGURE 29 SEACLIFF WAVES – SPECTRAL ANALYSIS (CULVER AND WALKER 1983)

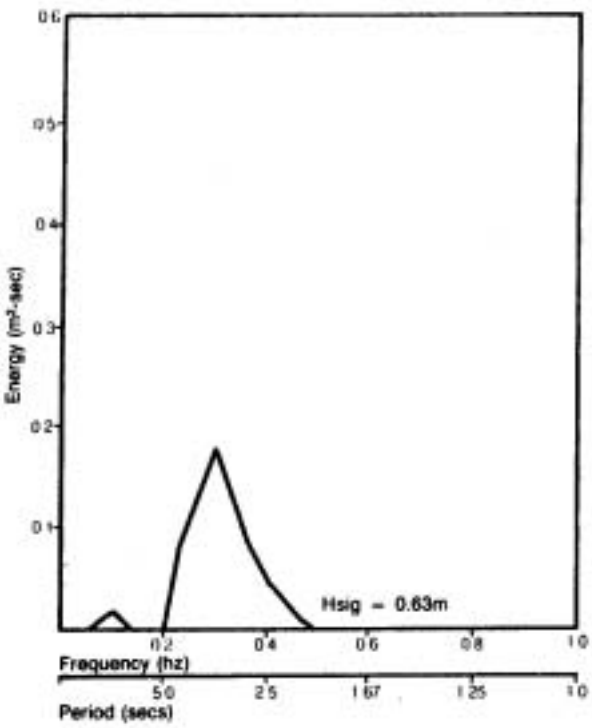


Figure 29A
Typical Energy Spectrum

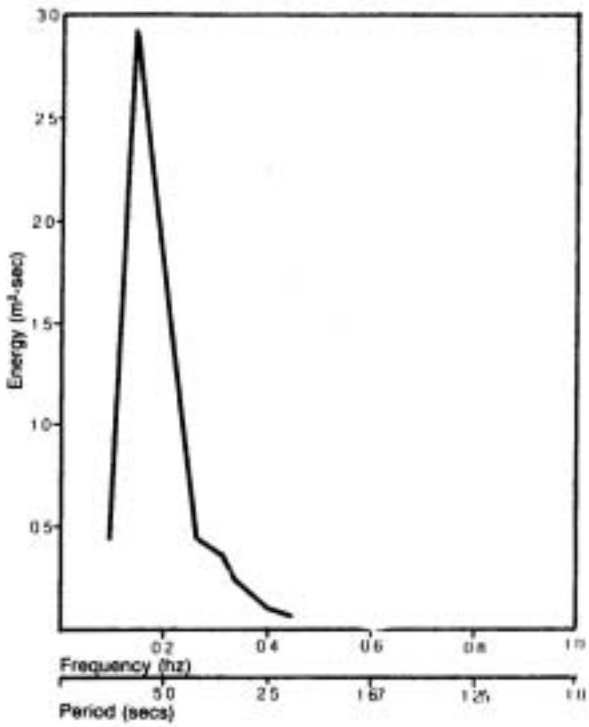


Figure 29B
Storm Energy Spectrum 30/8/81

The method allows for variable depth and fetch, as would be experienced during a wind direction change, and the energy is traced along the fetch as it grows and decays, until it arrives at the point of interest.

Culver (1970) compared wave hindcasts using this method with measured waves at the Grange jetty and obtained good comparisons, though only a limited number of hindcasts were done.

The method, as enhanced by Ijima and Tang, being based on a large amount of wave experience, is generally reliable and is widely used. Because it predicts wave heights and periods at only one point of interest, it is relatively quick, and this is an advantage if waves are to be hindcast over long time periods, such as for deriving long-term wave energy for sediment transport calculation. The method is best suited to simple input data where fetch and wind direction do not alter greatly as they do at Adelaide. It does not take account of refraction within the wave generation area, or of energy transfer with adjacent wave fields, though these are incorporated, to a certain extent, in the empirical constants in the formulae. Although refraction effects are not taken into account in the wave generation, they would nevertheless be applied in the nearshore area to obtain wave effects on the coastline. This was done in the 1970 University of Adelaide study in estimating the alongshore transport.

b) Culver – Walker Model 1980–83

The University of Adelaide wave study, sponsored by the Coast Protection Board, has included the development of a theoretically based wave model based on a method suggested by Gelci et al in 1956. The work was done by a postgraduate student, D. Walker, under the direction of Mr Culver, and is reported in Walker (1980) and in Culver and Walker (1983).

The method involves the solution of an energy balance equation:

$$\frac{\partial E}{\partial t} + Cg\left(\frac{\partial E}{\partial x} + \frac{\partial E}{\partial y}\right) = S$$

where $E = E(x, y, t, f, \theta)$ the wave energy
x & y are the spatial co-ordinator
t the time
f the wave frequency
θ the wave direction
cg the wave group velocity

The equation is solved over a two-dimensional space grid and the solution gives the full wave energy direction – frequency spectrum over the whole grid. The addition of suitable terms in the source function enables the theories of wave generation to be taken into account, and also bottom friction and refraction effects. The method is based on one developed by the Danish Hydraulics Institute for a study of the Gulf of Thailand, with the addition of a term to take account of bottom friction. The following terms are included in the source function:

Phillips wave growth term

Miles' wave growth term

wave breaking term

opposing wind, dampening effect

The model predicts the total wave field over the study area shown in Figure 30. A range of wave frequencies between 0.1 and 0.5 Hz were considered, and eight 45° direction bins were used. The method involved the solving of 48 equations at each of the 114 grid points for each time step.

The project had a significant research component as well as being intended to provide a useful wave prediction model. The technique had mainly been used elsewhere in the open ocean situation, and there was some doubt about its performance in shallow water, with limited fetch distances and where seas were slight in comparison with those in the open ocean.

An extensive number of hindcasting runs for comparison with recorded waves have shown that the method is potentially quite accurate, although discrepancies can still occur. Figure 31 shows a typical comparison between predicted and recorded waves during a storm.

The model is capable of including the effects of wave refraction, but this has not been used in the test runs done to date. Because of the coarse grid used, it was felt that this might have produced some odd effects due to local discontinuities (Culver and Walker 1983).

Wind data from the Outer Harbor Pilot Station was used in the first proving runs in 1980. However, because the model is sensitive to wind speed and direction, the University established three anemometers at Port Giles, Ardrossan and Aldinga. Walker (pers. comm., 1983) reported that these recorders had shown winds to be consistent over the area for the storm events analysed, and that the more recent model verification runs had used wind data from one or other of the aforementioned University anemometers.

The disadvantage of this model is that it gives very detailed output and that it requires large amounts of computer time. This makes it more suitable for predicting storm and maximum wave events than for long-term prediction of wave energy, as is needed for calculating alongshore sediment transport. A modified version of the model has recently been devised to offset this disadvantage (Culver and Walker 1983). This version of the model can provide wave information at a specific point much more quickly with only a slight loss in accuracy. Walker has tested the modified version against the recorded waves at Seacliff, and obtained wave energy predictions within 20% of those calculated from the measured waves, using the 3-hourly wind data. This test was done over a 10-month period with energy (H^2T^2) summed over that period. This accuracy is quite sufficient for prediction of sediment transport, the 20% limit of error being small in relation to the uncertainties in the empirical formula used.

FIGURE 30 UNIVERSITY OF ADELAIDE WAVE PREDICTION MODEL (SEA) LOCATION AND GRID

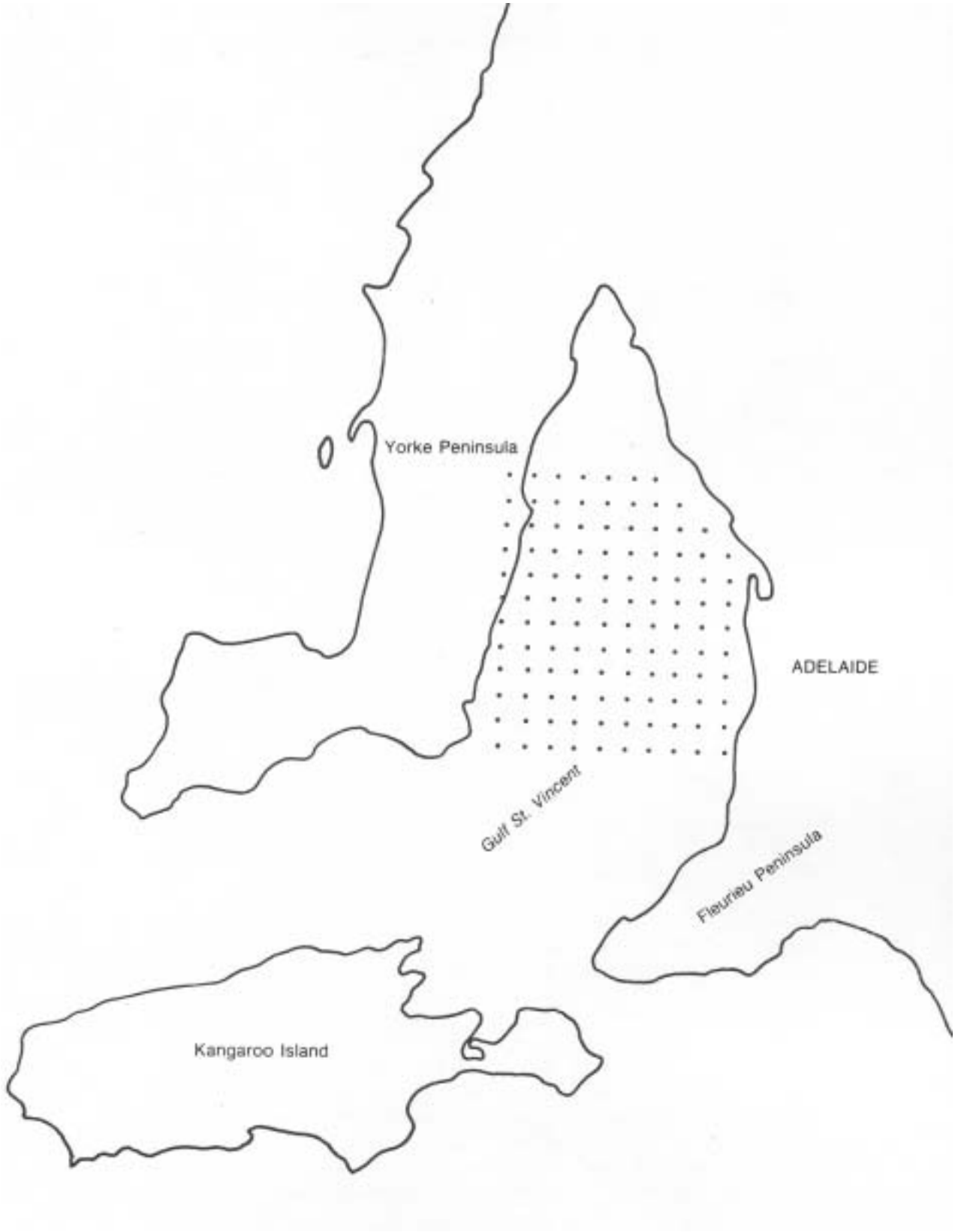


FIGURE 31 WAVE HINDCASTS VS RECORDED WAVES (SEA) (CULVER AND WALKER 1983)

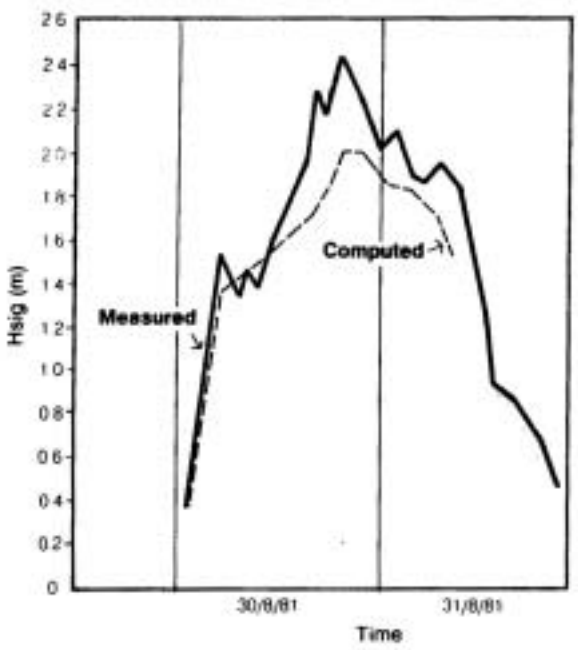


Figure 31A
Wave Height Comparison

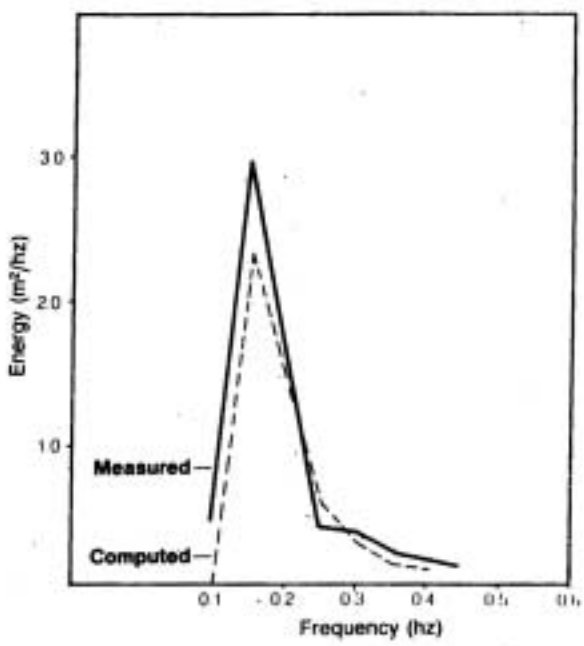


Figure 31B
Spectral Energy Comparison

The prediction of extreme waves is more important for marine structures than for beach and foreshore protection purposes. This is because of energy dissipation in the nearshore zone, and because the larger waves are depth-limited and break before reaching the seawalls or other coastal structures. Precise measurement of offshore waves is thus not very important for the design of nearshore coast protection works, though the order of magnitude needs to be known.

Wave Climate

A knowledge of the wave climate is important for construction activity at the coast, and especially for dredging, which can be very sensitive to sea conditions.

Figures 28A, 28B and 28C describe the wave climate. As can be seen from these, significant waves of 0.5 m prevail for approximately half of the time, and waves exceeding 1.5 m for approximately 6% of the time. Waves with significant heights of more than 1.8 to 2.0 m are rare, and only occur during storm events.

Figure 28B shows the duration of waves of various heights in relation to the number of occurrences. As can be seen, durations in excess of 20 hours apply frequently for waves between 1.0 to 1.4 m, but durations of the larger waves are much less, with a maximum duration of 12 hours having been recorded for a 1.8 m significant wave. The most common duration of these larger waves is less than 5 hours.

As shown in Figure 28C, wave period is in the range 0 to 8 seconds, with the longest period storm waves being mainly in the 6 to 7 second range. Wave periods in the range 2.5 to 4 seconds are the most common, though the main part of the overall wave energy is in those waves of periods 4 to 6 seconds (Culver 1970). Energy spectra derived from the Seacliff recorder are shown in Figure 29, for typical and storm conditions.

The summary of wave climate used for the Offshore Dredging Feasibility Study (see Section 5.2) provides a useful abbreviated description.

Average significant wave height 0.7 m

Average wave period 4 seconds

Seldom was significant wave height below 0.3 m

Seldom was significant wave height above 1.0 m

Maximum significant wave height recorded 2.5 m

Design Waves

Depending on the type of structure, different design waves are needed. For example, an isolated steel or concrete structure might fail as a result of forces generated from a single large wave, whereas a rip-rap seawall is influenced more by a succession of waves, each somewhat less perhaps than the extreme wave. The 'significant wave', being the average of the highest one-third waves, is generally the most useful, as well as being the wave size that most closely resembles the observed sea condition and its total energy. It is the most relevant measure in considering the usual range of coast protection structures.

Examination of wave records obtained during the storms of 1981 shows that the extreme significant wave height recorded was 2.5 m, though this occurred only once and for a short period. During the year, waves in excess of 2 m were recorded on several occasions. On the basis of these records, the engineering consultants for the Alternatives Study recommended use of an extreme significant wave of 3 m for design purposes. The return period of this sea condition was considered to be greater than 50 years.

The foregoing design wave estimate was for the offshore, deeper water situation. Because of breaking, shoaling and bottom friction loss in shallow water, the waves that reach the coast are less than this. This is particularly so in the Adelaide situation where the offshore and beach slopes are very flat. (The offshore slope is approximately 1:100 or less, and the beach slope is approximately 1:35.)

The consultants for the Alternatives Study calculated significant wave height for a range of inshore seabed contours, as shown in Table 2. The calculations were based on a deepwater significant wave height of 3 m, on a combined refraction and shoaling coefficient of 1 (ie negligible effect), and on the significant wave height being limited to half the work done by Kimura and Iwagaki (1978), who showed that, for very gentle seabed slopes, random waves break when the depth is twice the wave height. The wave heights in Table 2 are based on a design water level of AHD 2.7 m.

TABLE 2 SEABED CONTOUR VS SIGNIFICANT WAVE HEIGHT (KINHILL STEARNS AND RIEDEL & BYRNE 1983)

Seabed Contour (AHD)	Storm Water (Depth)	Significant Wave Height
+ 1 m	1.7 m	0.9 m
0 m	2.7 m	1.4 m
-1 m	3.7 m	1.9 m
-2 m	4.7 m	2.4 m
-3 m	5.7 m	2.9 m
-4 m	6.7 m	3.0 m

The design wave will thus vary with the amount of sand in front of seawalls or other structures, and will increase if beach levels drop. Greater heights apply to offshore structures such as offshore breakwaters and the heads of groynes than to seawalls. Kinhill Stearns and Riedel & Byrne suggest a design wave for offshore breakwaters in the range 2.5 to 3.0 m, and 3.0 m for the heads of

groynes. Their suggested design wave for seawalls is 2.0 m, which allows for a beach drop to 1.0 m AHD, approximately 1 m below the present beach fronting seawalls. Larger temporary beach drops than this may well occur, but could be expected to be associated with an increase in the size of the offshore bar, with consequent wave height reduction by wave breaking on the bar.

The probability of higher waves in deep water can be estimated using the Raleigh distribution and observed wave distribution curves given in CERC (1977). These indicate larger waves with the following probabilities based on a 3.0 m deepwater wave:

Probability 0.1 (ie one wave in 10) – wave height 3.3 m

Probability 0.01 (ie 1 wave in 100) – wave height 4.6 m

Extreme wave – in the range 5 to 6 m

The extreme wave would be most affected by the local shallow water conditions and can be expected to be less than the above figure, which is based (CERC 1977) on observations at coastal stations rather than on the Raleigh distribution.

Wave Refraction

The calculation of wave refraction is an important part of the process of calculating sediment movement, and also in predicting where wave energy will be focussed along the coastline. Waves that have been predicted or obtained from a wave recorder must be subjected to a refraction analysis to determine their direction at the wave-break point and their energy, which is a function of the height at this point. Accurate refraction calculation is critical for estimation of alongshore sand movement.

The usual process, as adopted by Culver (1970), is to derive refraction coefficients for sets of wavelengths and wave incident directions at each part of the coastline being considered. These coefficients are then applied to the offshore waves, and alongshore wave energy calculated separately for each set of wavelengths. These are then summed to obtain the total alongshore energy over a particular period. The process is a complex one, but one which has been well adapted for computer solution. Culver used a slightly modified version of a method developed by Wilson (1966). The refraction coefficient is the ratio of the wave height inshore (at the breaking position) to the offshore, deepwater, wave height.

The refraction package used by Culver has been refined as part of the recent Wave Study to use more detailed water depth information, and to use a slightly different method to overcome some problems associated with the earlier method (Culver and Walker 1983). In the revised computer program a method suggested by Abernathy and Treloar (1977) is used. Rays representing the wave orthogonals are refracted seaward from the point of interest to obtain the angular relationships and the refraction coefficients. The 380 m x 380 m bathometric grid used extends to 30 m water depth. The effect of refining the grid is uncertain and has not yet been tested. The effect may be small because most of the refraction occurs in the shallower water, where depths are influenced by the state of the tide and possibly also by short-term changes in bottom topography. This uncertainty in nearshore refraction represents one of the greatest sources of error in the calculation of alongshore wave energy, and hence in sediment transport.

Both the earlier and the revised method apply refraction analysis to separate sets of incident wavelengths. Culver (1970) analysed waves of 2, 4 and 6 seconds, and the recent study (Culver and Walker 1983) used five steps covering a range of wavelengths between 2 and 10 seconds. This separation within the analysis is necessary because waves of different lengths are refracted differently. The wave energy is summed following the separate analysis.

The Culver report (1970) considered the variation of refraction along the coastline, but only applied this to calculation of wave energy at one point in the study area, this being at Grange. The more recent University work has calculated and applied refraction at each of several segments into which the coast has been divided for the purpose of the sediment model.

As a part of the Alternatives Study for the Board, Kinhill Stearns and Riedel & Byrne carried out wave refraction calculations on waves of 6 seconds period only, these being the highest energy waves to

be expected with any regularity during a severe storm. Two locations, North Brighton and Point Malcolm, were considered. Their results indicated that refraction is not an important factor in reducing wave energy for Adelaide, or in concentrating or dispersing energy along the coast. Culver (pers. comm.) has advised that his more extensive refraction analyses for the 1970 report showed significant energy concentrations along the coast.

In making their observations on this, the consultants noted that energy reduction was likely to occur for other reasons – notably because of bottom friction across the shallow, wide bars offshore in the northern part of the study area, and due to friction in the seagrass meadows in the south. Possible erosion due to loss of seagrass and consequent greater wave energy reaching the coast is discussed elsewhere in this report.

During a storm, parts of a coast are damaged more than others due to the concentration of wave energy by refraction caused by variations in the nearshore seabed. Sometimes this results in an inadequately protected portion of the coast sustaining less damage than one with more substantial protection. Such lack of damage can give a false sense of security and can result in delay in providing necessary protection works. It is important to realise that the effect will apply differently for waves from different directions, and that it will change with changes in the nearshore bottom topography. The surprising lack of storm damage to the coast in the vicinity of the Henley jetty and swimming pool during the 1981 storms can probably be explained by the refraction phenomenon.

Wave Energy

Calculation of wave energy is fundamental to an understanding of how waves affect beaches and to what extent.

As a first order approximation the energy in a single wave and the energy flux reaching the shoreline from a train of waves are simply related to the wavelength, wave height and wave group velocity as follows:

$$E = \frac{\rho g L H^2}{8} \quad \text{and} \quad P = \frac{\rho g H^2}{8} \cdot Cg$$

where E = wave energy per unit width of wave

P = wave energy flux
ρ = density of water
g = gravity
L = wave length
H = wave height
Cg= wave group velocity

Since the wave height is proportional to the square of the wind speed and the energy is proportional to the square of the height, the energy is roughly related to the fourth power of the wind speed. Because of this, a large amount of the total wave energy occurs during storms when the higher winds prevail.

The present study has included no new work on the calculation of wave energy except insofar as this has formed a part of the associated wave study being undertaken by the University of Adelaide. The University work has included examination of the spectral energy distribution (see Figure 31B) and calculations for testing the sediment prediction model over short time periods using recorded waves.

Culver (1970) carried out a detailed investigation of the wave energy climate, and the findings remain valid. Figure 32 reproduces in SI units a table from the Culver report. This shows the wave energy distribution for each of 6 years, by direction and wavelength. As can be seen from this, the larger part of the energy is in the 4 to 6 second storm waves, though a substantial part is in the 2 to 4 second period waves, which occur at other times. Figure 29 from the recent wave study clearly illustrates this. In the typical energy spectrum, the energy peak lies between wavelengths of approximately 2.5 and 4 seconds. In the storm spectrum the peak is between 5 and 6 seconds wave period.

The minor peak in the typical energy spectrum in Figure 29A represents the energy in the long period swell. It is small in relation to the total energy and can be safely ignored in the energy and sediment movement calculations.

The wave energy figures in Figure 32 and the annual resultants of these (Figure 33) apply to offshore conditions, and do not take account of refraction. Comparison of the energy resultants in Figure 33 with the resultants of onshore wind components (Figure 26A) shows that there is considerably less variation in the wave energy resultant directions than in those of the wind resultants. This is to be expected, due to the filtering effects of the fetch and bottom topography on wave generation.

As discussed in Section 2.5.2, on winds, the resultant wave energy direction has a northerly component, even though the resultant onshore wind component may be slightly to the south. Culver's analysis demonstrates this for 1956. The recent wind analysis, illustrated in Figure 26, shows that 1981 is the only other year known to have had an annual onshore wind resultant with a southerly direction. A wave energy summation has not been done for that period. However, the resultant wave energy for the year would almost certainly have had an alongshore component in a northerly direction.

Culver calculated refracted wave energy as part of his sediment movement calculation on an hourly basis, and then collated both the sediment transport and wave energy on a monthly basis for each of the 6 years he examined. However, no summation was done on the refracted energy on its own for comparison with the resultant obtained using offshore waves. No further work has been done on this.

2.5.4 Alongshore Sediment Transport

Discussion here is limited to alongshore sediment movement occurring within the surf zone and driven by wave energy. Other sediment transport mechanisms, which are likely to play a smaller role and which are less significant over short time periods, are discussed in Section 2.6.3.

In the past, many coastal structures have been designed and built in Australia and elsewhere without an adequate understanding of the prevailing alongshore sediment movement. It is understandable that many of these have as a result interacted with the natural beach system to cause more erosion problems than they have resolved. Experience has particularly emphasised the need for an adequate knowledge of alongshore sand drift.

Both the engineering design and the longer-term economics of strategies considered in this study show critical dependence on the alongshore drift quantities, over both long and short periods. For example, groyne spacing, beach replenishment quantities, and dredging at the Patawalonga channel are all fundamentally related to alongshore sediment movement.

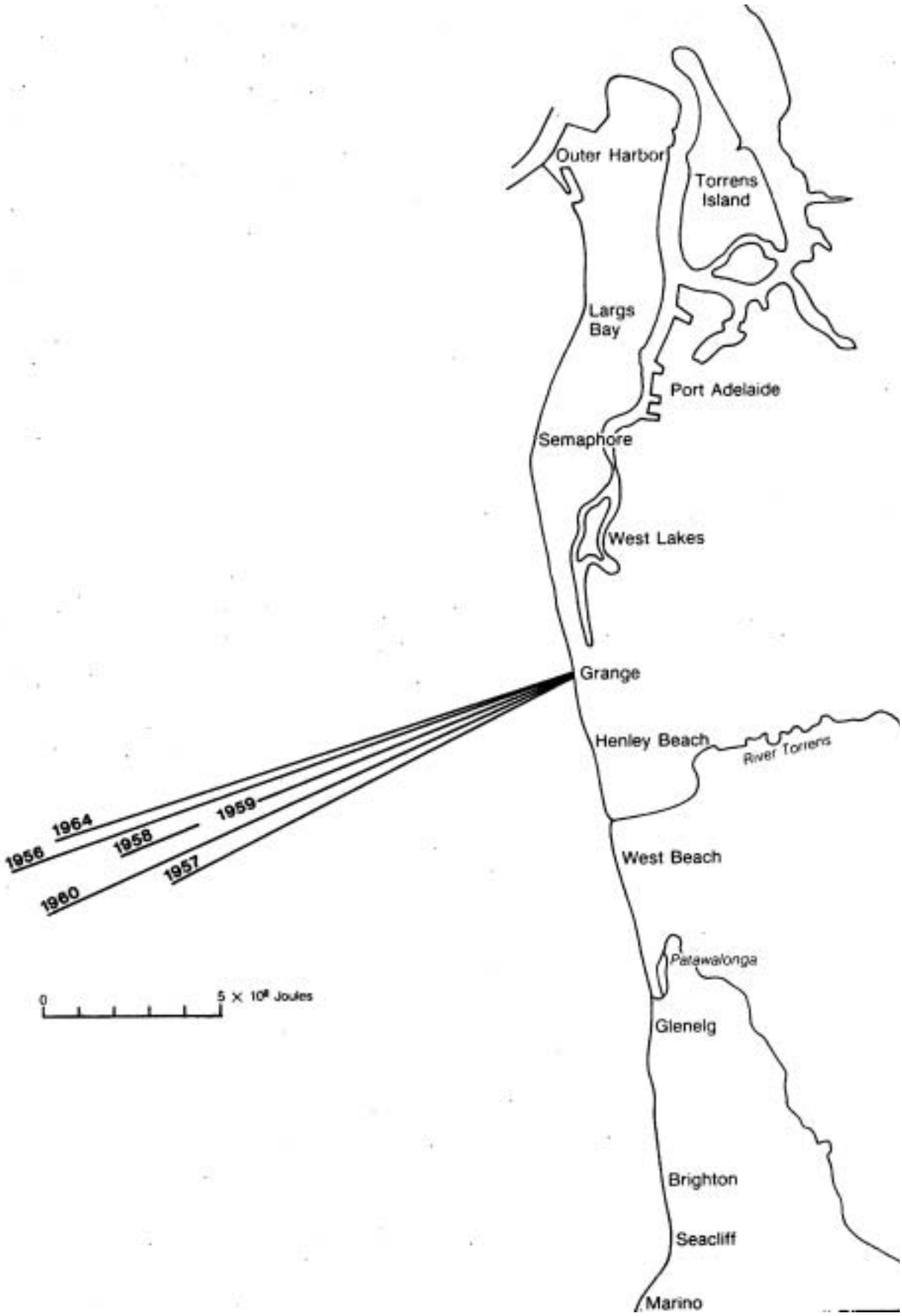
Measuring Sand Accumulation

The most reliable way to estimate the average annual rate of transport is to measure the accumulation at a barrier such as a groyne that extends across the active beach zone. Estimates may be somewhat high because the barrier usually provides a sheltering effect for waves in the 'opposite' direction. For example, southward movement of sand from the Outer Harbor/North Haven area will be small because of this sheltering effect, whereas northward sand movement into the area would be relatively unaffected. The same would apply at the only other suitable structure at Glenelg.

FIGURE 32 WAVE ENERGY TABULATION – GRANGE (CULVER 1970)

DIRECTION (Degrees)	WAVE PERIOD GROUPS (secs) 2-4	4-6	6-8	8-10	YEAR
220	103.24	321.53	5.06	0	1956
250	48.04	247.36	0	0	
280	33.71	176.15	0	0	
310	31.18	110.83	0	0	
220	74.59	304.67	10.11	0	1957
250	25.71	205.64	1.69	0	
280	30.34	104.90	0	0	
310	40.03	45.51	0	0	
220	68.69	260.00	15.17	0	1958
250	41.72	246.94	0	0	
280	48.46	87.23	0	0	
310	29.08	61.10	0	0	
220	78.80	174.46	0	0	1959
250	29.92	180.78	0	0	
280	25.28	46.35	0	0	
310	31.18	24.86	0	0	
220	93.55	321.53	53.52	0	1960
250	39.19	320.69	13.91	0	
280	33.71	71.22	0	0	
310	45.51	60.68	0	0	
220	127.68	237.67	10.11	0	1964
250	37.93	262.95	0	0	
280	40.45	128.95	0	0	
310	46.78	94.82	0	0	
	ANNUAL AVERAGE VALUES (For six years)				
220	91.02	270.12	15.59	0	
250	37.08	243.99	2.53	0	
280	35.40	95.66	0	0	
310	37.50	66.16	0	0	

FIGURE 33 RESULTANT ANNUAL WAVE ENERGY VECTORS



The accumulations must be large enough to be reliably measured taking account of the superimposed seasonal beach changes. They must also be made early in the life of a structure, before it has filled with sand and reached the bypassing stage. The Glenelg groyne reached this stage within a few years of its construction in 1964–65. The southern breakwater at North Haven, constructed in 1974, appears to have started bypassing, though this may still be small in relation to the rates of accumulation south of the structure.

Culver (1970) deduced a long-term average rate of transport by comparison of surveys of the accumulation at Outer Harbor with earlier hydrographic surveys. The southern Outer Harbor breakwater was built between 1903 and 1905. A 1966 survey was compared with 1875 and 1946 hydrographic surveys. An annual drift rate of 28,300 ± 11,500 m^3 was estimated from these. This is considerably higher than the rate that Culver calculated at Grange using wave energy. Culver also examined the accumulations at the Glenelg groyne and the Torrens River outlet, but was unable to deduce transport rates from these. A physical model was built for the Patawalonga Outlet, but this was of too small a scale to be used quantitatively.

In his estimate for Outer Harbor, Culver assumed that half the accumulation was sand and the other half seaweed, which comes ashore in large quantities in that area. Subsequent trial excavations in the onshore seaweed accumulation area in 1980 showed that the seaweed compacts to form only about 20% of the total volume. This suggests that the accumulation here might have been as much as 40,000 to 50,000 m^3/year, though it must be remembered that the accumulation rate here will be higher than the alongshore drift rate because of the wave sheltering effect of the breakwater, and may be affected by a supply of finer material from offshore.

The southern breakwater at North Haven could have been expected to cause measurable sand build-up over the past 9 years. This has occurred and is evident on the beach, but the measurements also show an unexplained erosion in the nearshore zone (see Section 2.6.3) and this apparent inconsistency has made the calculated quantities unreliable.

The present sand replenishment program has been in progress for nearly 10 years, and the quantities moved, together with the measured results, suggest an average rate for the 10 years of approximately 30,000 m^3, though it is difficult to separate this from the effect of sea level change. Figure 34 shows the quantities moved during the years of operation of the replenishment program. The average rate of removal between 1977 and 1982 of 33,000 m^3 a year from south of the Patawalonga groyne is of particular interest as this sand was taken partly with the intention of preventing bypassing of the groyne and thus reducing blockage of the boating channel. The channel improvement was not achieved by this means, and it was apparent that the sand quantities returning by natural processes were much larger than those being taken out. This rapid sand return would be partly due to restoration of the local beach equilibrium from within the general large deposit between the groyne and the jetty, and would not be entirely due to alongshore drift from the south.

One drawback of deductions based on either accumulations or on the trucking program is that allowance is not made for possible offshore losses, as may be occurring to a significant extent north of Glenelg, and that accumulation at structures is only relevant to places of similar wave climate, coastal alignment and sediment characteristics. The transport rate at Brighton would be quite different to that at North Haven – the coastal alignment, sediment availability, and sediment size being quite different.

There are thus obvious limitations to deductions based on the accumulation at existing structures. This applies both to use of the structures to estimate longer-term quantities, and to their use to test and calibrate the mathematical sediment movement model.

Also, because of the unusually high variability in local wave energy directions, and consequently in annual sand transport, measurement can only provide a rough long-term figure. It cannot provide information on the actual rate for a single year, nor on the range of variation that might be expected. It does not provide useful information on the gross, to and fro, movement as is needed to estimate channel-dredging quantities.

FIGURE 34 BEACH REPLENISHMENT

73/74	74/75	75/76	76/77	77/78	78/79	79/80	80/81	81/82	82/83	Removed per Year (10 Year Average)	Placed per Year (10 Year Average)	
14 335								1 172		29 842		Outer Harbor
							11 250			1 125		Largs Bay
			7 600	3 750		30 000	44 841	11 250 15 289 22 500	2 285 7 500 14 925	18 054		Semaphore
	5 735						4 772			1 050		Grange
	5 735			3 750			4 772 11 250	11 250	14 925		5 168	Henley Beach
	8 601										860	
	8 601	11 250 15 000		11 250 6 744 22 500	30 000	22 500	22 500			15 034		West Beach
		15 000 12 422		6 744 22 500	30 000	22 500	22 500				10 916	
14 335 10 422		11 250 2 546	7 600	11 250		22 500		22 500 15 263 1 172	7 500 22 885		14 924	
10 422	40 547	12 422	1 500	30 000		22 500 48 750	30 000	28 875 15 263	23 250	22 755		Glenelg
14 335	40 547 2 073	3 730 2 216	1 500 3 634	8 625 22 500 30 000	4 379	3 750 30 000 48 750	44 841 11 057 30 000 990	10 125 28 875 15 289	14 250 23 250		39 472	Brighton
	2 073	3 750 2 546 2 216	3 634	8 625	4 379	3 750	11 057 990	10 125	14 250	6 638		Marino Port Stanvac

Cubic metres (×1 000)
200 100 100 200 300 400 500
Removed Placed

The separate effects of alongshore transport and sea level change are discussed further in Section 2.6.3.

Alongshore Drift Calculation – Mathematical Models

Various empirical models have been devised to predict the alongshore sediment movement component of wave energy flux. The most commonly used of these has been that set out in CERC 1966 and revised in CERC 1973 and 1977. Before discussing the local application of these models, it is worth noting that alongshore drift calculation of this type is prone to large errors, and should be regarded only as supplementary to estimates obtained by observation and measurement of actual beach changes.

Following wave energy and wave refraction analysis, as previously discussed, the alongshore component of the wave energy flux is calculated as follows:

$P_1 = E.Cg.\ \mathrm{Sin}\alpha\ \mathrm{Cos}\alpha$

Where: P_1 is the longshore energy flux

E is the wave energy at the breaker position

α is the wave approach angle at the point of breaking

Cg is the wave group velocity (the wave phase velocity can be used as an approximation)

Depending on the units used, this will usually be either in feet-lb/s or joules/s. As will be seen from the following, calculation of sediment transport involves the use of constants, and these vary depending on the measurement system used. To avoid confusion, the old imperial system will be used here, even though this is no longer in application in Australia. This is convenient because both Culver's work and the revised US publications updating the method use the imperial system.

The experimental relationship recommended in CERC 1966 and used by Culver (1970) was:

$Q = 210\ P_1^{\ 0.8}$

Where: Q = drift rate in cubic yds/day

P_1 = alongshore energy flux in millions of feet-lb/day/linear feet of beach

Culver expressed this in units per hour, and used it to calculate drift quantities for each hour for the years that he analysed. He applied it at one point only, at Grange, and necessarily assumed the results as applying along the coast between Brighton and Outer Harbor.

The more recent version of this formula (CERC 1973 and 1977) is:

$Q = 7.5 \times 10^3\ P_1$

Where: Q = drift rate per year

P_1 = alongshore energy flux in feet-lb/sec/linear feet of beach

It is noted in CERC 1977 that considerable judgement is required in applying the equation. It is suggested that the average difference between field results and the formula derived therefrom is at least 28% of the predicted value and possibly higher. Subsequent work, noted in CERC 1977, could indicate that actual transport rates might differ from calculated values by even more than this.

Notwithstanding these possible inaccuracies, the method is still probably the best available, and the only way to obtain a reasonable estimate of the relative movement along the coastline in both directions. Its accuracy can be much increased by adjusting the constant according to known local accumulations where this information is available.

The CERC method assumes linear wave theory, and takes no account of sediment characteristics or of the actual mechanisms involved in the entrainment and transportation of the sand. More recent work, by Longuett Higgins (1970, 1972) and Bijker (1967), adopted a more theoretical approach. In the case of the former, this led to logical derivation of a part of the constant, though

the resulting formula was similar to the empirical CERC one. The Bijker method, which is based on bed shear due to wave and current motion, has not been used or assessed for the local Adelaide situation. It requires more information than is currently available for Adelaide.

The basic CERC method assumes a length of coast of one alignment and an infinite supply of sand. The real situation can be quite different, with transport rates varying with changes in coastal alignment to the dominant wave direction, and with sand supply being restricted for a variety of reasons, such as the existence of hard horizons in the beach zone and the presence of coastal defence works.

Culver's (1970) calculations followed the basic CERC method in ignoring special inhibiting or accelerating factors. The annual net northerly transport quantities obtained were in the range of 10,000 to 22,000 m^3 for the 6 years examined, with the gross (northerly plus southerly) quantity being in the range 15,000 to 27,000 m^3.

Alongshore transport was calculated on an hourly basis using hourly wind data and summed for each month. It should be noted that these gross annual figures are derived by summation of the monthly rates calculated by Culver, and that the gross obtained by summation on a daily or hourly basis would be higher and more accurate.

The gross transport is relevant to maintenance of a channel at the Patawalonga and consequently to the cost comparison of protection strategies that may reduce the alongshore transport and hence maintenance dredging costs. This is covered in Section 5.6.2. Adjusting the presently supposed net transport rate of 30,000 m^3/year using gross/net ratios deduced from Culver's work suggests an average gross rate in the range 40,000 to 50,000 m^3/year. However, as noted, Culver's annual gross figures were summations of net monthly figures and this must give a low result. In the absence of better information, a figure of 80,000 m^3/year was given to the consultants for use in the Alternatives Study, to be used as a basis for their dredging estimates.

Culver carried out model tests to determine the effect that seawalls might have on the transport rates. The results of these were useful, though not entirely conclusive. The tests suggested that the presence of a seawall within reach of the waves would result in an increase in the drift rate. They also indicated that the longer-term result might be a reduction in drift as beach levels fall in front of the structure, exposing underlying strata and reducing the amount of available sand. Culver did not attempt to adjust his calculated quantities to take account of this.

The early version of the CERC formula has been checked against the recent version to test the change in the constant and the power relationship, as these apply to the incident wave energies and transport rates obtained and used by Culver. Depending on the assumptions made as to how the energy is distributed throughout the year, the revised formula suggests results ranging from slightly less than Culver's (on the unrealistic assumption that wave energy is evenly distributed throughout the year) to approximately twice Culver's figures (assuming that the energy and transport is concentrated into periods representing a quarter of the year). Although very rough, this suggests the possible order of variation within use of two versions of the same formula. It demonstrates that the figures calculated by Culver may not be inconsistent with the presently supposed annual average transport rate of 30,000 m^3 a year.

The local calculation of sand movement is being refined as a part of the aforementioned wave study recently carried out for the Board under Mr Culver's direction. The present model is similar to that previously used. As noted in the foregoing discussion of refraction, the refraction is calculated by a slightly different method. The main refinement to the model is in the division of the study area into 13 separate segments and in the transport rate being calculated separately for each of these. This enables the differing wave incident conditions and variations in coastal alignment to be taken into account, as well as other special conditions that may apply in a particular segment. The method also has greater potential to predict areas of erosion or accumulation. Because of the more complete picture that the refined model will give, it should be possible to check it more closely against the observed coastal change, and consequently to calibrate it and improve its accuracy. The work is still in progress, and it is not useful to provide details of discussion of the initial test runs at this stage.

When fully developed, the modified version of the Culver–Walker wave model could be used either with half-hourly wind data (obtainable from the Adelaide Airport with some processing costs) or with the 3-hourly data that Walker (1983) has shown to give predicted wave energy within 20% of that calculated from measured waves.

The greatest difficulty with local calculation of sediment transport is undoubtedly the lack of field information to calibrate and check the model. As discussed, movement over longer time periods cannot be reliably estimated and the best estimate available is at Outer Harbor, which is not typical of the more critical part of the coast. Measurement over intermediate time periods of 2 to 5 years could be made if a new totally occluding structure were built for the sand to accumulate against, but would not be reliable because seasonal changes could be large in comparison to the quantity being measured. This effect could be reduced by taking measurements several times each year, and averaging them, though the survey effort would be large.

Sand tracer experiments can provide useful qualitative information about inshore sediment movement, but accurate determination of long-term sediment transport volumes is not yet possible by this means (CERC 1977).

South Brighton Alongshore Tracer Experiment (Blue Sand)

A sand tracer experiment was conducted in conjunction with the 1981–82 summer sand replenishment program at Brighton. The purpose of the sand tracer experiment was to detail general features of sand dispersion such as direction of movement, depth of mixing and lateral spread and, if possible, to determine an estimate of littoral drift rates. The study location is shown in Figure 35. It includes the main Brighton beach replenishment area, and the adjacent beaches onto which the sand would be expected to move. The beach is approximately 30 m wide at mean high water, and a sandy bottom extends seawards to the seagrass meadows that commence approximately 200 m offshore.

FIGURE 35 BLUE SAND TRACER EXPERIMENT – BRIGHTON

The underlying assumption used in all experiments of this type is that, after a representative tracer is introduced into the beach system and time allowed for mixing, observations of the movement of the tracer particles will be representative of all the particles in the beach system.

In March 1982, a truckload (approximately 10 tonnes) of blue dyed sand was placed on Brighton beach opposite Shoreham Road. The sample, taken from Port Malcolm, a replenishment source beach, had been treated by the Australian Mineral Development Laboratory (AMDEL) using a dyeing technique successfully developed by the University of Sydney and applied to sand tracer studies by the NSW Public Works Department on Cronulla beaches.

During the first week after deployment, a series of experiments including sand dispersion and nearshore current (within the first 200 m of the shore) measurements were conducted. Following that, a program was initiated to determine the long-term behaviour of the dyed sand field. Three transects, two north and one south of the deployment site, were established. The northern transects were located at Cambridge and Alfreda Street, 500 and 1250 m north, respectively. The southern transect was located at Portland Street, 450 m south. The northern transects were monitored initially at weekly intervals and subsequently at fortnightly intervals, between April and October 1982.

The supra-, inter- and sub-tidal regions along each transect were sampled at 5 m intervals using 32 mm OD polycarbonate tubing, which was inserted into the sand to a consistent depth between 350–400 mm. The core samples were inspected on-site for presence of blue dyed sand particles and then forwarded to AMDEL for dyed sand concentration analysis to allow subsequent correlation with wave and wind data.

Analysis of the results is not complete, particularly correlation analysis of blue sand levels, beach levels and nearshore levels with wave wind data from the Seacliff wave recorder. The results are reported in a Branch Technical Report (Petrusevics 1983). However, preliminary analysis indicates a number of features. These are summarised as follows:

1. The behaviour of tracer material is strongly time- and site-specific. For example, after initial contact by the incoming tide, the fine fractions of the dyed sand rapidly moved northwards extending about 200 m in about 2 hours. Thereafter the dyed sand stayed relatively intact, being overlaid by sand entering the study area. Thus, principally because the beach in the study area was in an accretion phase, the dyed sand showed little movement away from the site of deployment. Had the beach system been subjected at this stage to an erosion stage the initial displacement of the dyed sand would have been different.

   Table 3 shows the surface concentration of dyed sand as a function of distance along the coast from the point of deployment. The figures quoted refer to surface concentration on day one after deployment.

TABLE 3 BLUE SAND TRACER – SURFACE CONCENTRATION AND DISTANCE ALONGSHORE (PETRUSEVICS 1983)

Distance (Metres)	Surface Concentration (Blue Grains/75 mm Square)
175	40
225	15
350	5
450	2

   Rate of movement along the beach was determined by noting the time of arrival at sites other than Cambridge and Alfreda Street, northwards of the point of deployment. Table 4 shows typical results.

TABLE 4 BLUE SAND TRACER – RATE OF ALONGSHORE MOVEMENT (PETRUSEVICS 1983)

Location	Distance (Metres)	Elapsed Time Since Deployment (Weeks)
Cambridge Street	500	1–4
Alfreda Street	1,250	8–9
Minda Home	2,100	15–16
Eton Road	3,000	26–28

2. The extent of offshore movement of the dyed sand was limited to the landward side of the seagrass line, which, in this locality, is about 200 m offshore. Preliminary estimates indicate that at the Cambridge Street transect 70% of the dyed sand was located within 100 m of the rock protection line and at Alfreda Street 80% of the dyed sand was within 60 m of the rock protection line. The limited offshore presence of the dyed sand highlights the importance of the corridor defined by the seagrass meadows and, presumably, the high water mark on the beaches as a zone where most of the littoral movement probably occurs.
3. During the period of observations, the extent of southerly movement was negligible. Intermittently, and corresponding to prolonged periods of northerly winds, trace levels of dyed sand were detected along the Portland Street transect. However, generally, during this period, unidirectional sediment movement to the north occurred.

   The extent to which dyed sand techniques may be used to determine littoral drift rates is yet to be resolved. The technique outlined by Kadib (1972) could not be fully tested, because the level of dyed sand could not be monitored for a period corresponding to dyed sand first being detected until all traces of dyed sand had disappeared from that transect. The latter criterion was not achieved during the period of observations.

   This may be attributed to the Adelaide metropolitan coastline being a low energy coastline in general, and specifically because the 1982 weather conditions were calmer than usual.

   For example, the depth of mixing of the dyed sand reflects, in an indirect way, the energy of a given coastline. The depth of mixing along Brighton beach was measured to be about 38 ± 17 mm, whereas Kadib (1972) reported 85–110 mm for the Mediterranean, and Gordon (pers. comm., 1982) quotes depth of mixing up to 1 m for the east coast of Australia.

   The technique is labour-intensive and analytical analysis is expensive, so the value of the method as a quantitative tool is questionable, for the Adelaide metropolitan coastline at any rate. The main use of this technique is as a qualitative tool to highlight paths of movement along beaches, leakage through semi-permeable groynes or bypassing around headlands.

Barge-Dumped Tracer (Red Sand)

Dumping sand close inshore by bottom-opening barge was one of the methods of beach replenishment considered as part of this review. DMH owns two split-hull barges and a bucket dredge, which could be used to fill the barges with suitable offshore sand from a reasonably protected site. The method therefore deserved consideration both as a possible supplement to the present trucking program and, with additional barges, as a method of replenishment on a larger scale. However, the method depends critically on how close inshore the barges can dump, and on whether or not the dumped sand would be moved ashore by natural beach processes. A tracer experiment was commenced in February 1983 to investigate these aspects.

FIGURE 36 RED SAND TRACER EXPERIMENT – BRIGHTON

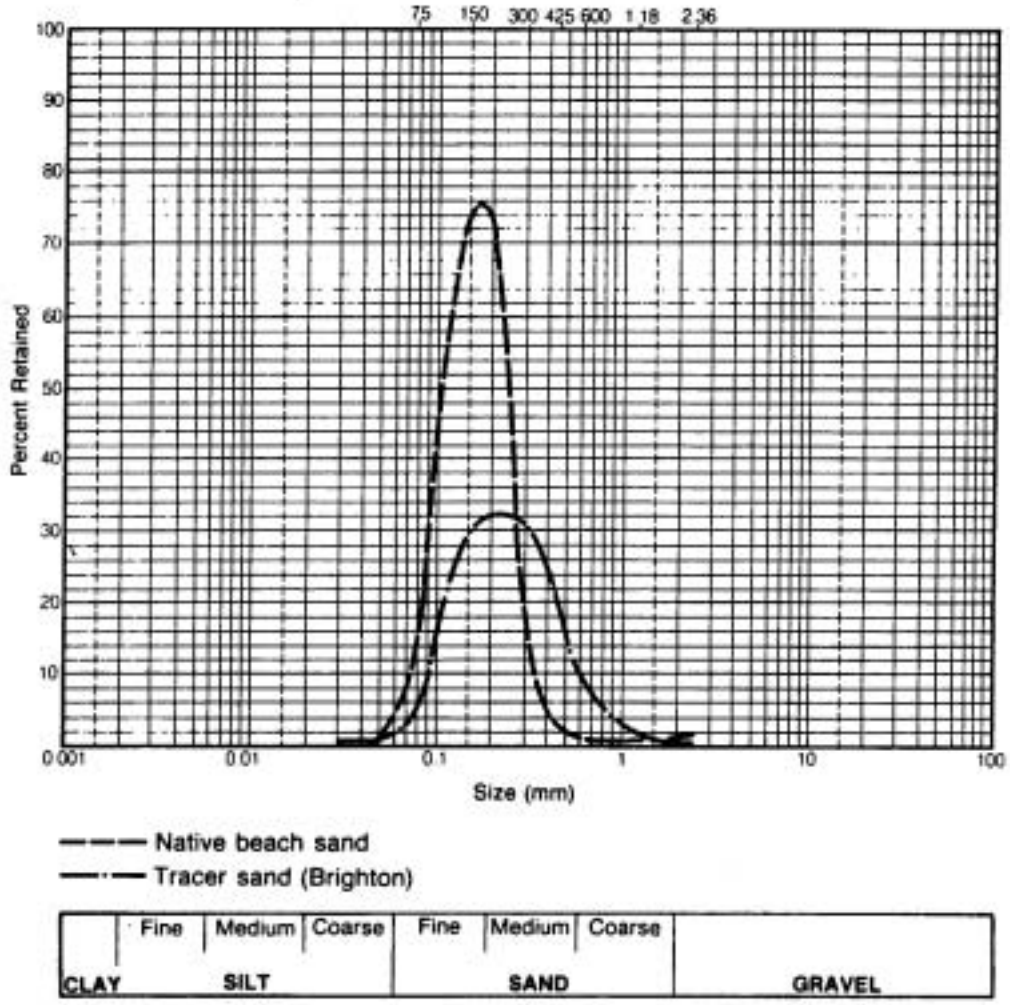


Figure 36A
Gradings of Native and Tracer Sand

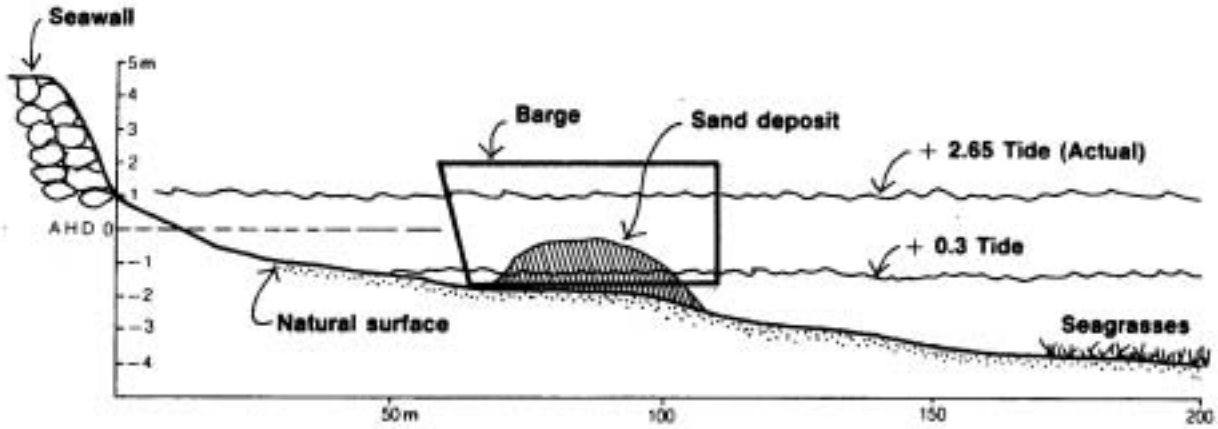


Figure 36B
Beach Profile at Dump Site

PHOTOGRAPH RED SAND TRACER EXPERIMENT, BRIGHTON

PHOTOGRAPH OFFSHORE SAND SAMPLING

Four hundred tonnes of a natural red, 'plastering' sand was obtained from a commercial sand quarry and loaded into the 'Sainsbury', a 500 m^3 split-hull dump barge owned by DMH. The barge sailed to Brighton and dumped the sand 80 m north of the jetty and as close inshore as it could safely reach. The 400-tonne load was equivalent to 250 m^3, a half load for the barge.

The tracer sand was slightly coarser than the native sand, as can be seen from the grading curves in Figure 36A. However, a substantial part of the material was of similar grain size to the beach material, and this portion could be expected to behave in a similar manner to a later replenishment material. The specific gravity of the sand was 2.66, which is virtually identical to that of the natural beach sand.

Ideally, dyed beach sand should have been used, but the cost would have been prohibitive for the quantity needed.

The barge sailed in towards the coast, dumped its load and reversed out, remaining at right angles to the coast. The heap of dumped sand thus approximated the length of the hopper (25 m) and was aligned at right angles to the beach. Figure 36B shows a profile across the dumpsite.

The sand was dumped on a 2.65 m tide, which is near the top of the normal range of high tides. Predicted tides vary between approximately 1.6 m and 2.7 m with occasional predicted high tides outside these limits. At a commonly occurring low tide of 0.4 to 0.5 m, the seabed under the dumped sand would be at a depth of 0.1 to 0.3 m at the shallowest, landward end. Allowing for the bottom slope, the tracer material was within a range of water depths between approximately 2.8 m and nearly zero. The dump was located mainly on the upper part of the seaward slope of the offshore bar, and slightly onto the bar itself. The centre of the dumped heap was approximately 100 m from the toe of the rock protection behind the beach.

The movement of the sand was initially monitored by vertical aerial photography at twice-weekly intervals, and later at fortnightly intervals. Shallow coring and terrestrial surveys were carried out at regular intervals.

There was an initial immediate rapid northward, alongshore movement of the fines from the upper part of the dump. The red colouring reached South Glenelg within a few hours. Subsequent leaching out of fines was slow, and there was no later marked discolouration of the water.

During the 2 to 3 weeks immediately following the dumping, there was a progressive flattening of the heap, combined with an inshore movement onto the bar and a northward movement along the bar. The sand appeared to be moving from the inshore part of the dump, with the seaward part showing little influence by coastal processes other than a slight flattening. Movement shoreward of the offshore bar was not detected.

Monitoring in mid April followed the first stormy conditions since the dumping. The main part of the red sand had moved slightly seaward and had been covered by approximately 30 to 40 cm of native sand. That part of the sand that had earlier moved shorewards and along the bar appeared to have been mixed with the native sand and was not detectable.

No monitoring was done over the winter months. The most recent check, in October, found only a small quantity of the red sand. This was found at the original dumpsite, layered with native sand in the part closer to shore, and in a single 50 mm layer in the part further offshore. The layering comprised a surface layer of 25 mm of red sand with a layer of similar thickness of native sand sandwiched between this and a lower layer of red, also of approximately 25 mm in thickness. No traces of red sand could be discerned either inshore or seaward of the dumpsite or along a transect 25 mm to the south. A trace of red silt was found in a transect 25 m to the north, in the top 100 mm and adjacent to the dumpsite.

It must be concluded from this that the tracer was mostly dispersed and mixed with the native sand by the winter storm waves, to an extent that would only have been possible if the sand were in the active beach zone. By implication, this suggests that the tracer would have been eventually moved ashore, first under post-storm wave conditions, and later by the flatter summer waves. The absence of tracer sand in the less active zone toward the seagrasses suggests that there was very

little, if any, seaward movement. Had sand moved seaward, it should have been easier to detect, because there would have been less mixing.

It has therefore been shown that sand could be dumped on the offshore bar at suitable times if shallower draft barges were available, and that such sand would become part of the beach system. However, beach replenishment using this method would be unlikely to be practical because of the tidal constraints and the need to purchase the special barges. The experiment has not excluded the possible hazard to seagrasses, which would almost certainly apply if this method were used for a major beach replenishment. The seagrasses are known to be important for fisheries production, but their importance in avoiding coastal erosion remains unknown.

2.6 COASTLINE CHANGE AND THE SEDIMENT BUDGET

The purpose of this section is to review and bring up to date information about changes in the Adelaide coastline position and to relate these changes to a 'sediment budget' – insofar as this is possible with present knowledge about the coastal processes.

Both coastline change and the sediment budget will be considered only for the period since European settlement (the past 150 years approximately), with emphasis on the past 50 years, during which aerial photography has been available and for which more survey information exists. The longer-term geomorphology of the coastline development is described in Section 2.2.1, and is not considered here.

Culver (1970) covered most of the earlier material, though did not include a detailed assessment of change using aerial photography, and did not have as much beach profile information as now exists. This new information is used to check Culver's work and to carry it a little further.

2.6.1 Change in Coastline Position

Information Available and Methods Used

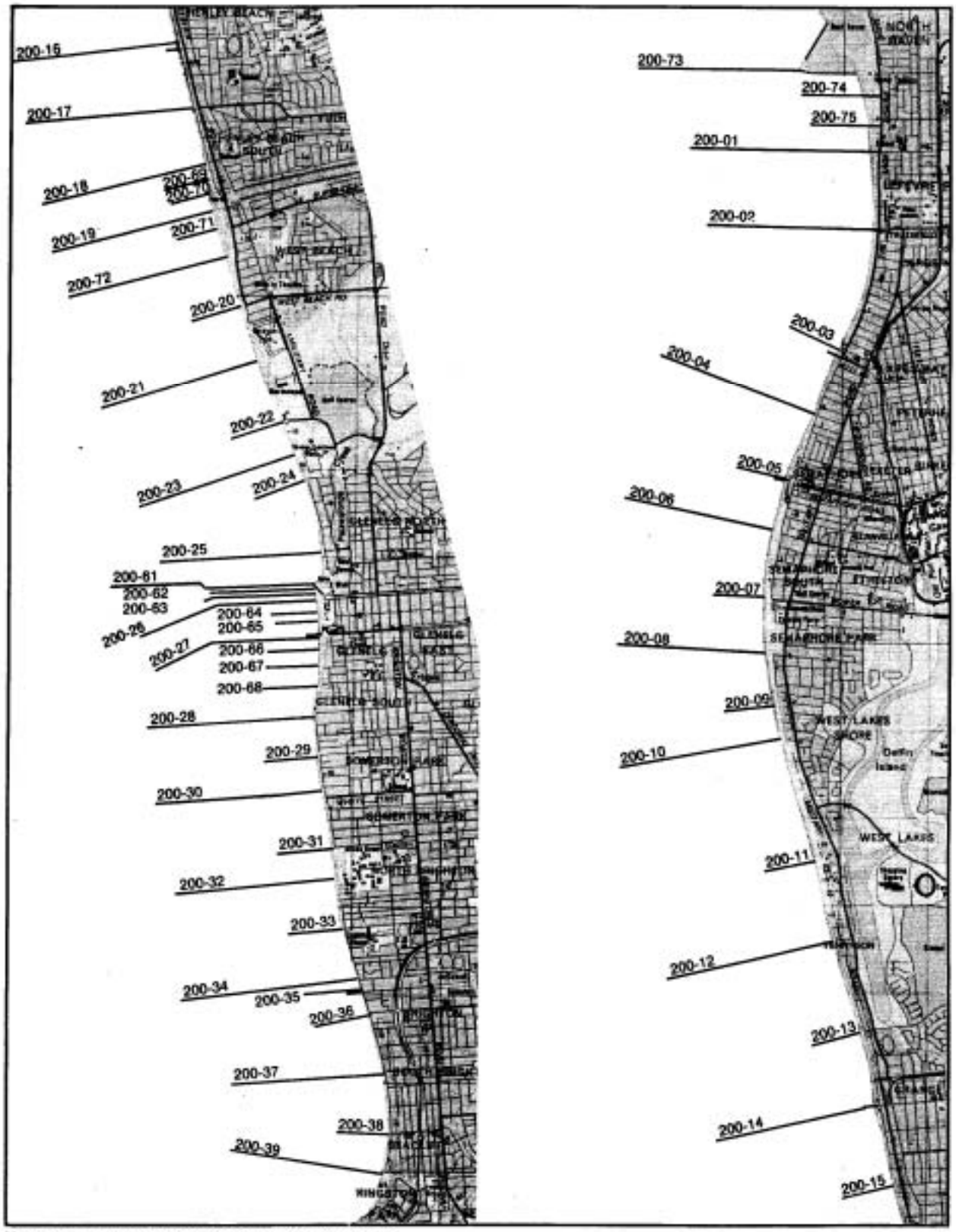
The most reliable method of measuring coastline change is to compare accurate topographic surveys, especially where these extend seaward to include soundings. This type of survey is available at Outer Harbor and Glenelg, and to a lesser extent at Brighton, North Glenelg and the Torrens Outlet. Such surveys, which were carried out for engineering projects, can be compared with recent, special coastal surveys, such as those included in the Branch's beach profiling program.

Another usual source of information is early Government surveying done for land allocation. These surveys are variable in the precision with which they identify and locate high water mark (HWM) and, surprisingly, give less attention to this for the Adelaide coast than for many other parts of the state's coastline. An 1876 survey shows the HWM in the Brighton – South Glenelg area at positions up to approximately 10 to 15 m seaward of the present HWM or seawalls. The accuracy is uncertain and little can be deduced from this except that there appears to have been no significant erosion at Seacliff south of the rock protection; that such erosion as may have occurred between Seacliff and the Brighton jetty has been reclaimed through the beach replenishment program; and that there has been a historic loss of beaches between the Brighton and Glenelg jetties.

A large amount of precise surveying was done by the Engineering and Water Supply Department for sewer design purposes, mainly between 1900 and 1930. Although this shows very clearly the contours with the coastal allotments, it does not extend far enough seaward to give much useful information about coastline change. The present study has not attempted to use it for this purpose.

Beach profiles were measured as part of the 1966–70 University study and have been measured for the Branch by the Department of Lands since 1975. The profiles are survey lines approximately at right angles to the beach, and extending from seafront roads or property across the beach and nearshore zone. The 1966–70 University work included 36 beach profiles and three more precise jetty profiles.

FIGURE 37 BEACH PROFILE LOCATIONS



Note: Most profiles surveyed bi-annually since 1975

Figure 37 shows the profiles presently being measured in the study area. Thirty-five lines extend 1 km or more to sea, across and beyond the active beach zone, and the remainder, as indicated in the figure, extend only to low-tide wading depth. Most of the lines are surveyed twice a year, in winter and summer. However, a selective reduction has been made over the past 3 years, partly to contain the cost. Such reduction is appropriate now that baseline average profiles have been established over several years at most of the stations.

Present use of these profiles for measuring trends in coastline change is limited because of the large seasonal variation in beach shape, because of the limited accuracy in deeper water, and because it is economically impractical to have a close grid of profile lines over the full length of the Adelaide coast. Profiles are spaced at approximately 0.5 km, with closer spacing at a few special places. Alongshore movement of coastal features (such as a temporary embayment, sand lump or offshore bar configuration) can thus move across a profile line, giving a spurious result. A longer period of record is needed before the profiles generally become reliable for measuring longer-term coastline change. At this stage, the profiles have been mainly used in providing information on seasonal beach movements – for design of coastal structures and management of the beach replenishment program.

The beach profiling program will be fully described, together with results to date, in a Branch Technical Report to be prepared in the near future.

Aerial photography provides the most useful means of identifying progressive coastline change, though it is inadequate for reliable estimates of quantities, except perhaps where marked recession of a dune face has occurred, such as at West Beach. Although low-altitude photography can be used for quite accurate surface levelling, a test at a trial metropolitan station (near the Torrens Outlet) showed that its use was not practical as an alternative to beach profiling. Existing photography was flown at too high an altitude to give the precision needed, and the cost of special very low altitude flights, together with the photogrammetric analysis, would be prohibitive. It was concluded that aerial photography was most effective when used for measuring horizontal change, in conjunction with terrestrial levelling to provide more detailed information at selected places.

As part of the review study the Branch compared aerial photography of the study area between 1935 (the earliest available photography) and 1981 (Moulds 1982). The photographs were interpreted for the years 1935, 1949, 1953, 1961, 1972 and 1981, and the coastline position was plotted using a Stereo Zoom Transferscope (ZTS) for the years 1953, 1961, 1972 and 1981, the later photography being to a higher standard and more useful than that from the earlier years. Photograph scales varied between 1:5000 and 1:16000.

Only horizontal change was assessed – the seaward dune face or beach berm, which is clearly discernible on most photographs, being used for this. Roads and property boundaries provided good control. This, together with the enlarging, rotating and translation features of the ZTS, enabled accurate plotting of the coastline position from the photography onto 1:2500 maps.

The 1953 and 1981 photography provides a particularly good comparison because both were flown soon after major storm events of approximately the same severity, with the dune face escarpments being sharply defined on both sets of photographs.

A significant limitation of the photographic study was that useful comparisons could not be made where there were seawalls – the change in beach level being undetectable (though changes in beach width could sometimes be assessed).

Space precludes more than an overview of this study here and the inclusion of samples only of the mapped information. Figures 38 and 39 are typical examples, which respectively show the progressive erosion of the West Beach Recreation Reserve frontage, and the seaward progradation adjacent the River Torrens outlet.

FIGURE 38 COASTLINE CHANGES AT WEST BEACH

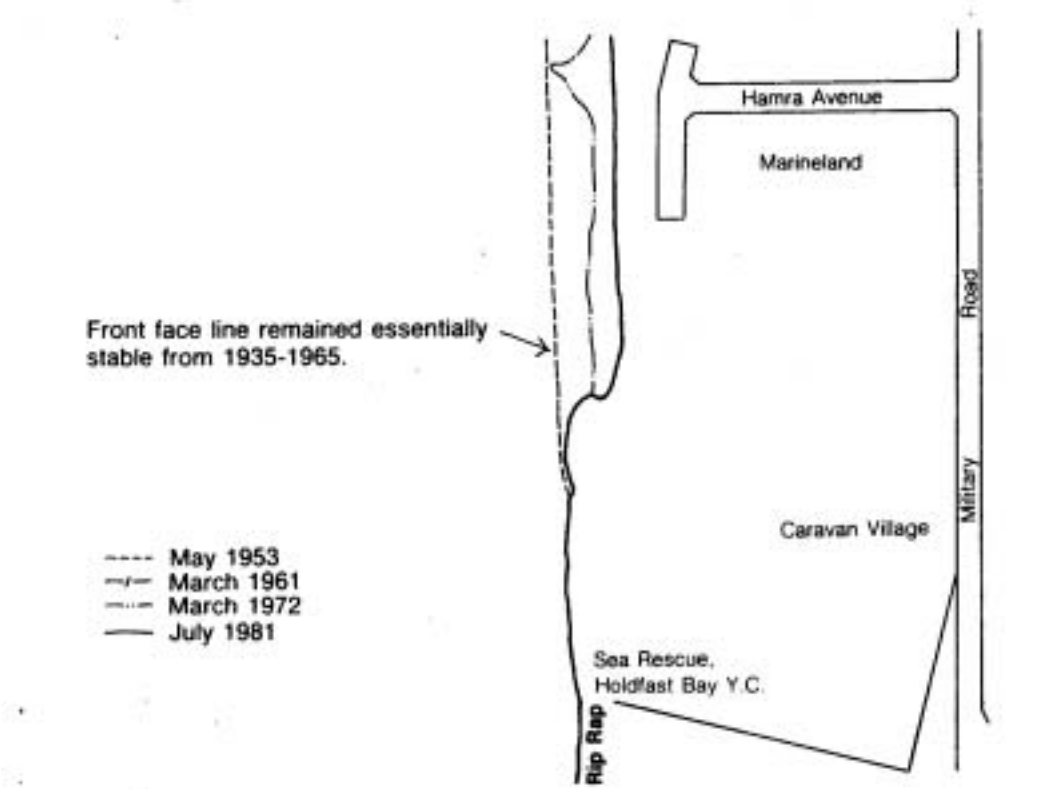
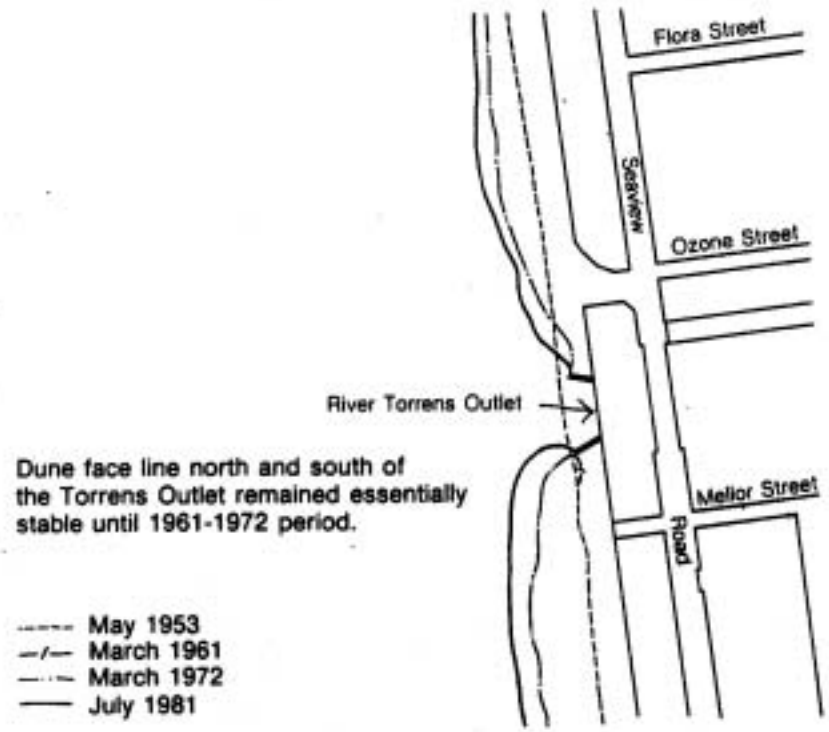


FIGURE 39 COASTLINE CHANGES AT THE TORRENS OUTLET



The photographic study showed that the coastal change (mostly retreat) has accelerated in the period since 1961, after having been relatively static before that. This confirms observations made by Culver (1970). Most of the observed change has been in the middle section of the study area (Somerton to Henley), and can clearly be related to construction of the groyne at Glenelg and to the progressive building of seawalls.

The progressive loss of the Brighton and Seacliff beaches is discernible from the photographs, though the most obvious changes have been the retreat of sand dunes at Minda and West Beach and the structures and sand accumulations at Glenelg, North Haven and Outer Harbor. It is interesting to note that significant build-up at the Torrens Outlet (Figure 39) only started in the 1960s (about the same time as rapid erosion elsewhere), despite the outlet having been constructed much earlier, in 1937. With this exception, the changes can readily be related to the building of coastal structures including the progressive construction of seawalls. The effects of remedial measures, such as dune planting at West Beach, are discernible. The noted changes are all consistent with a predominant northward littoral sediment movement, and a steady loss of sand from the southern areas.

In the northern part of the study area, the photographs show a variable seaward movement of the coastline. A steady seaward growth of the dunes between Grange and Point Malcolm is evident, with a significant widening of the beach at Point Malcolm. The greatest area of progradation was at Outer Harbor and, in recent years, to the south of North Haven. Although large seaward movements have occurred in this area, dunes have not formed – the new land being low-lying and separated from the beach by low sand and seaweed ridges.

In the Alternatives Study, Kinhill Stearns and Riedel & Byrne used the beach profiles and the aerial photography, in conjunction with a theoretical method developed by Swart (1976), to determine the width of the 'active beach zone'. They found good correlation between the results from the different methods – showing that the width of the zone varied from approximately 200 to 500 m, and generally approximated 300 m.

They related width of the zone to the prevailing coastline change, and suggested that the zone was widest where there was a surplus of sand and where accretion was occurring, and narrowest where there was a sand deficit. Their concluding observation was:

> At profile 006 (Semaphore) the beach zone is accreting and this zone is wider than might be expected for a stable beach.
>
> At profiles 010 (West Lakes Shore) and 012 (Tennyson), the beach is in a stable state and if anything is accreting.
>
> For profiles 014 (Grange) to 018 (Henley Beach south), the beach appears to be stable. Sand appears to be moving past this area with little change occurring to the beach itself.
>
> From profile 072 (West Beach) southward, the beaches are eroding. It is inferred from this interpretation that the active beach zone here is narrower than it would be if there were enough sand supply. In other words, if sand is added to these southern beaches, much of it will be used in creating a wider active beach zone. It will not necessarily show as a wider beach amenity until the active zone has reached a stable width.

The final observation is particularly relevant to the design of a beach replenishment scheme.

These findings are consistent with results of volume comparisons made from the profiles by Culver (1970) and by the Branch since then. These show loss of sand from profiles in the Brighton area, little change in the Henley/Grange area, and gains in the northern part of the study area.

Coastline movement at the more critical or informative areas is discussed in detail in the following sections in relation to the sediment budget.

2.6.2 Nearshore Changes

Offshore change was examined by Culver (1970) by comparing special 1969 soundings (these extended the University beach profiles seawards to 3.2 km to sea) with hydrographic surveys carried out in 1868 and 1948. The 1948 survey gave implausible results and had to be rejected as either being in error or of inadequate accuracy for the purpose. Culver's main conclusions from examining the 1969 and 1868 information were that erosion had generally occurred inshore along the entire coast length, except at and north of Largs Bay, and that there had generally been an accumulation offshore north of Broadway. He noted that the coast to the 4-fathom (7.3 m) depth had experienced a net erosion of 5 million cubic yards over the 100-year period (ie an average of approximately 40,000 m^3 a year). Part of this would have moved northwards out of the study area before the Outer Harbor breakwater was built.

Culver included analysis at depth bands of 1 fathom, with separate findings for each. However, the accuracy limitations in using conventional hydrographic surveys for this purpose are such that much weight cannot be attached to these. He deduced from his results that 'once sand is placed offshore it does not readily come back'. He drew attention to the dependence of his conclusions on the accuracy of the surveys and noted that effects of the then more recent works, including all the structures discussed in this report, could not be separated from the general changes over the 100-year period.

Bathometric changes were also examined as part of the Gulf St Vincent Nearshore Sediment Dynamics and Sedimentation Study (Hails et al 1970) described in Section 2.2.4. 1980 soundings were compared with 1946, 1948, 1957, and 1958 hydrographic surveys, over coast widths and water depths of approximately 4 km and 10–12 m respectively. The comparison showed an overall accretion trend, with belts of accretion and erosion between Marino and the Torrens Outlet, erosion between Glenelg and Henley Beach, and accretion north of this.

It has not been possible to reconcile the more recent study with Culver's work – the former showing overall accretion within a 4 km wide strip, and the latter, erosion within a 3.2 km strip, though local trends within the general area are identified in both with some agreement. Neither has it been possible to verify the reliability of either study. More recent experience with the Branch's beach profiling program, and particularly comparison of profiles with the recent hydrographic contour survey at North Haven, casts even more doubt on the use of soundings to measure offshore sand volume changes with reliability.

At this stage, it must be concluded that offshore change has not been conclusively quantified, and that this remains an area of uncertainty in the Sediment Budget.

2.6.3 The Sediment Budget

The sediment budget approach involves a comprehensive look at areas of sediment loss (usually sources for other areas), at gains (usually losses from other areas), and at the transport mechanisms linking these. This can often enable missing information to be deduced, and can provide explanations for local phenomena. While it is useful to the present study, it is limited by uncertainties about certain important local processes. This makes the sediment budget approach difficult for quantitative use, as will be evident from the following text, which describes attempts at this.

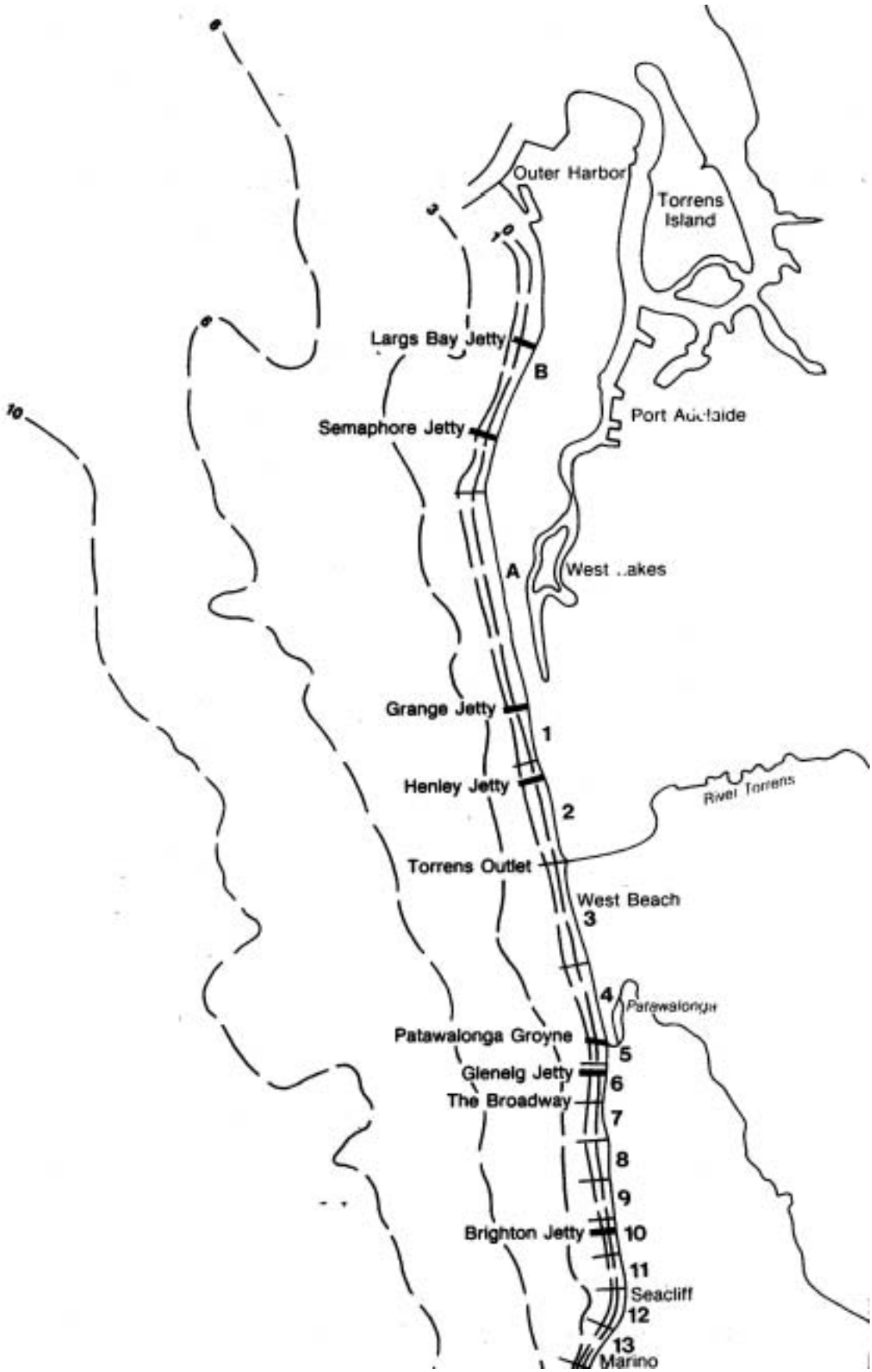
Losses of nearshore seagrasses may be affecting the coast between Somerton and Henley. Although bearing on the sediment budget, this factor is discussed separately in Section 2.7.

Sediment Compartment Boundaries

It is useful to determine the sediment movement for the whole area being examined, and also for sub-compartments within it.

The study area is itself a fairly self-contained compartment, with little alongshore movement across either its south or north boundaries.

FIGURE 40 THE SEDIMENT TRANSPORT MODEL SUB-COMPARTMENT BOUNDARIES (CULVER AND WALKER 1983)



The landward boundary should be chosen to include beaches and a sufficient width of sand dunes to allow for their contribution during storm events. Landward losses across this boundary must be taken into account.

In common with most similar studies (CERC 1977) the seaward boundary is the most difficult to decide, because of the limited knowledge of sediment movement in this area. The shorter-term, more rapid sediment movements occur close to the coastline, usually within the band of shore-parallel bottom contours (CERC 1977). Either the seaward limit of these contours or the landward boundary of the seagrasses would appear to be the most useful seaward boundary for the beach sediment budget.

Transport does occur further offshore due to the combined effect of tidal currents and the influence of surface waves. The Nearshore Sediment Dynamics and Sedimentation Study (refer to Section 2.2.4) deduced shoreward sediment movements from examination of seabed forms and grain size analyses. The rate of such movements, whether into or out of the coastal compartment, is unknown, but likely to be small and mainly involving finer sediment than that in the active beach zone. Where appropriate, such sediment movements will be discussed as inputs or losses across the seaward boundary.

On a continuous sandy beach, such as the study area, sub-compartments are not sharply defined, and the boundaries discussed here should not be given undue weight.

For the littoral drift model (see Section 2.5.4), thirteen sub-compartments were selected on the basis that each was considered to act in a uniform manner for its entire length. These boundaries, as shown in Figure 40, take account of changes in coastal alignment and of interruptions to alongshore drift such as are caused by the groyne at Glenelg and the Torrens Outlet. These 13 compartments cover the coastline between Marino and Grange, and exclude the northern part of the study area. Two additional compartments are considered here. These are: the straight length of dunal coast between Grange and Point Malcolm; and Largs Bay, between Point Malcolm and the southern North Haven breakwater. These are denoted ‘A’ and ‘B’ respectively to avoid confusion with the numbers used in the sediment transport model.

Consideration of the small, artificial sub-compartment between North Haven and the Outer Harbor breakwater has little relevance to this review, and is excluded.

Sediment Sources

It has already been noted (Section 2.2.1) that the sand in the study area is mainly a finite relict deposit, which has been transported to its present position by wave action during the Holocene sea level rise. Although present sediment contributions from outside the immediate system are thought to be very small, contributions from storage within the system (eg sand dunes) are significant. Present sources of sediment, from both outside and within the study area, are reviewed under the following headings.

a) Cliff Erosion and Alongshore Transport

There are no cliffs within the study area, and those to its immediate south are of a hard material and are contributing little sediment, despite being under active erosion. The closest cliffs that may be contributing a significant quantity of sediment to the coast are at Witton Bluff, Port Noarlunga, and south to Sellicks Beach.

These southern eroding cliffs undoubtedly contribute to the beaches in their vicinity, though the erosion rates are small and the quantity would not be great. The cliffs are composed of a variety of materials, much being in the clay and silt grain sizes. The finer component of the cliff erosion product would be removed in suspension (and would be deposited further to sea), leaving only the smaller coarse fraction to supplement the beaches.

Southern beaches (eg at Christies Beach and Hallett Cove) are closely aligned to the predominant wave direction (Culver 1970; McDonald, Wagner and Priddle 1982), and there would be very little net sediment movement along them. However, some northward spillage could be expected at times of strong southerly wind. Such sediment would be moved around the headlands and into the

deeper water off the wave-cut platforms of this coast. Some sediment could be moved northwards in deeper water, though the rates of transport could be expected to be less than those for transport along a sandy beach.

A small amount of sand may also move along the base of the cliffs, despite there being very little evidence of this – the small beaches between Hallett Cove and Kingston Park are pebble beaches and contain very little sand. The wave energy striking the cliffs is high and the predominant angle oblique, so any movement would be rapid. In 1970 DMH built a small boat ramp across the upper part of the rock platform at Kingston Park, and monitored this for sand build-up. J. R. Sainsbury (pers. comm., 1982) advised that sand build-up had been rapid, and that the small 'groyne' had filled in 2 weeks. This suggests, very roughly, a maximum rate of up to 5000 m^3/year, assuming that the transport rate represented the annual average rate and that all the sand moves along the upper, landward part of the rock platform.

Culver (1970) compared grain size and calcareous content for the southern beaches and those in the study area. His analysis showed the average median grain size for the southern region (from Hallett Cove to Sellicks Beach) to be 0.35 mm, which is considerably larger than the average of 0.20 mm that he obtained for the beaches of the study area. He also found that the calcareous content of the southern beaches was considerably less than that for the beaches between Kingston Park and Outer Harbor (2% vs 9%). A more recent analysis of beach sand accumulation at Port Stanvac (Fotheringham 1982) showed that this sand was closer to that of the study area than had been indicated in the Culver Report. Median grain size south of the Port Stanvac mound was 0.214 mm and that north of the mound was 0.176 mm. The calcium carbonate percentages were 10.1% and 14.0% respectively. Fotheringham estimated the annual accretion rate at Port Stanvac to be in the range 7,500 to 11,200 m^3, and suggested that most of the material was probably from alongshore drift, with a small amount coming from offshore. It would seem likely that at least a portion of the alongshore drift would continue to the north, notwithstanding the 'trap' effect at Port Stanvac.

The foregoing information is inconclusive though indicates that there may be a small but significant sediment contribution from the south. The quantity is probably less than 5,000 m^3, but may be as much as 10,000 m^3 a year.

b) Sand Supply from Rivers

Rivers are traditionally considered the main source of beach sediment, but this may not be generally true. As continents have weathered and flattened, the quantity of river-borne coarse sediment supply to coasts has decreased. Dam construction has contributed further to reducing the supply. Exceptions are small rivers through sandy catchments, and mountain streams that discharge directly to the coast. It is now thought (CERC 1977) that most new beach material is provided by erosion of coastal cliffs, and that the rising sea level is contributing to this.

The few small rivers and drainage channels at Adelaide do not carry much coarse sediment. The little coarse material that they carry is mainly deposited en route and cleared away by councils during drain maintenance.

The fine suspended sediment supplied by these rivers and drains settles out further to sea and in mudflat and mangrove areas. Long-term accumulation of fine material in these areas may have an indirect effect on the adjacent sandy beaches, though this is probably offset by the relative rise of sea level.

The effect of inland drainage on the beach sediment budget is therefore assumed to be insignificant.

Stormwater discharge, however, does cause temporary water turbidity and pollution, which may be having an effect on nearshore seagrass growth, with a consequent effect on adjacent beach sediments. This is discussed in sections 2.6.2 and 2.7.

c) Sediment From Offshore

The physical complexity and the difficulty in making accurate measurements make this the area of most uncertainty in any sediment budget. The local situation is no exception, but this may not be important, because local contributions from offshore are likely to be slight. An added difficulty in assessing this component is that virtually all research work on offshore sediment transport has taken place off exposed coasts, and where seabeds were mainly sandy without vegetation.

Komar (1976) makes the observation that 'In the few published budgets that have been developed for various coastal areas, it has been concluded that the offshore source is of minor importance in comparison to the other sources.' As will be discussed, local circumstances suggest that any offshore contribution to the budget will be small. However, the overall budget is a small one, especially in respect to sources, and even small contributions may be significant.

It should be noted that discussion here is limited to sediment movement between offshore and the beach-bar-dune system, and does not include seasonal and storm event changes within this latter system. These do not affect the overall sediment budget, except insofar as large storms may remove sand from this system. This will be discussed under 'Sediment Losses'.

Offshore sediment is subject to transport either by waves alone or by sea currents assisted by the oscillatory motion induced by waves. The general view (King 1972; CERC 1977; and others) is that nearshore sediment will move landwards under wave influence alone, though this movement may be very slow. There are several US examples of beach replenishment sand that has been dumped offshore and which has made virtually no landward movement over several years.

Grain size is significant, with the coarser material generally moving shoreward and the finer material seaward. It is notable (Hails et al 1983; Culver 1970) that local offshore sediment is generally finer than the nearshore and beach material, though the area offshore of the Onkaparinga River appears to be an exception to this.

The Shore Protection Manual (CERC 1977) indicates that significant wave-induced transport will occur where maximum wave-induced water particle velocities exceed 0.5 feet/s (0.15 m/s approximately) at the seabed. Typical local storm waves will cause these bottom velocities in water depths up to 30 m (ie virtually throughout the gulf waters). The movement will depend on the depth of water, the sediment grain size, and the presence of tidal or other currents. Sediment disturbed by wave action can be more readily moved by the prevailing current, and will be moved mainly in the direction of the current. Unfortunately, little is known about near-bottom currents in the study area.

Limited measurement using current meters (FIAMS 1979) and drogue buoy tracking by the Branch suggests that tidal current directions are closely aligned with the bottom contours, and are mainly parallel to the shoreline (see also Section 2.4.2). There is little evidence of onshore/offshore tidal current movement. The drogue buoy experiments show that near-surface tidal currents are modified in direction and strength by winds in excess of 5 m/s (approximately 10 knots). It is not known whether these effects extend downwards to the seabed. The current meter and drogue buoy experiments indicate normal maximum tidal currents of 0.4 to 0.45 m/s (0.77 to 0.87 knots), and maximum, wind-assisted, near-surface currents of up to 1 m/s (1.9 knots approximately). Drogue buoy tracking has indicated that the currents closer to the shore are less, reducing to maximum values of 0.2 to 0.3 m/s outside the breaker zone.

Despite its limitations, measurement to date indicates an absence of onshore/offshore currents, and thus supports the view that the offshore zone does not contribute significantly to the sediment budget. Sediment movement and bottom current directions have been deduced from sea bottom bedforms and grain-size analysis (Hails et al 1983) as described in Section 2.2.4 and shown in Figure 12. The bedforms suggest onshore/offshore bottom current directions, though this has not yet been confirmed by measurement of currents. The study identified sand ribbons extending landward from distances of nearly 2 km to sea. These were considered by the authors to indicate a limited offshore supply of sand.

From the foregoing, it must be concluded that sediment movement is occurring in the offshore zone, and that this movement may be shorewards. However, the extent is unknown. Given the limited depth of sediment (including its absence in places), the smaller offshore grain size, and the considerable seagrass cover, it would seem that a significant contribution to the sediment budget is unlikely. Possible exceptions are at places where nearshore seagrass is being lost, or where the nearshore waters are shallower, and where there is less seagrass cover. An offshore contribution in the North Haven area may explain the finer beach sand that occurs north of the Largs Bay jetty.

There is no evidence of onshore movement or sand accumulation in areas of seagrass loss – rather the reverse, as is discussed under 'sediment losses'.

As has been noted, an alongshore transport may be occurring to some extent in deeper water around the cliffed coast south of the study area. For the present purpose, this will be considered together with the component moving along the rock platforms. A combined figure of 5,000 to 10,000 m^3 a year is assumed.

d) Sand Supply by Erosion of Sand Dunes

Over the past approximately 5,000 years, the main sand source for the study area has been from the sand dunes at Brighton, Seacliff and South Glenelg. These dunes are still there, but are now built on and separated from the beach by seawalls.

Other dunes along the southern half of the study area would also have contributed increasingly as the supply from those further south was cut off. The dunes have been progressively isolated by seawall construction, with only two relatively undeveloped areas, at Minda Home and at the West Beach Recreation Reserve, remaining. Significant erosion has occurred at both these locations. Until 1973, erosion was also occurring to dunes at the Glenelg Sewage Treatment Works, but this sand supply was eliminated by construction of seawalls in 1974.

The construction of seawalls has not only cut off supply from the dunes behind them, but has created erosion pockets at ends of the separate lengths of seawall. These 'end-effects' have resulted in rapid erosion (with consequent sand supply to beaches) from the adjacent dunes. This has only partly offset the loss of sand supply due to the dunes nearby being isolated from the beach. The end erosion has prompted extension of the seawalls to solve these new erosion problems.

The only significant end effects are now at the Minda and West Beach dunes, and at the northern extremity of the rock seawalls, near Grange Road. The latter seawall ending is provided with regular replenishment sand to lessen the effect and to avoid the need for further northward extension.

Erosion of the West Beach dunes has been studied by Culver (1970) and in the Coastal Management Branch by Beare and Moulds (1981). Culver provides dune profiles at the Glenelg Sewage Treatment Works showing up to 20 m of erosion of the dune face between 1940 and 1968. He considered that this area had provided much of the sand to maintain the northern movement and to keep up beach levels. He also noted the simultaneous inland sand loss by deflation from the dunes.

In assessing the contribution of sand dunes by erosion of the front face, consideration should be given to inland wind-blown losses, partly from the beach, partly from the front dune face and partly from within the dunes. At West Beach, this inland loss is evident as a landward migration of the rearward edge of the dunes. Comparison of aerial photographs (Moulds 1982) shows only a small average movement of the rear edge after 1967, though shows movement at rates of up to 15 m a year before this. There appears to be no significant landward sand loss at the Minda dunes. Use of Culver's (1970) deflation figures indicates that up to 2,700 m^3/year may be lost inland from the 850 m length of dunes at West Beach. It is impossible to estimate how much would come from the beach, which is wet for part of the time, or from the dune face, where turbulence and funnelling could be expected to increase the rate. Much of the sand from the beach is trapped on the dune face, and that from within the dune could be expected to be trapped by dune vegetation, depending on the extent of this. The average rate for the length of dunes would be less than the possible 2,700 m^3/year.

At West Beach. The Branch's volume estimates are based on recession of the front dune face as measured by aerial photography between 1972 and 1981, and using a typical survey profile through the dunes. The estimated annual average quantity of sand derived from the dune face over the 9 years is between 10,000 m^3 and 15,000 m^3. The average total recession during the period was 35 m. A slower erosion rate nearer to 3 m/year was found to have applied between 1977 and 1983, this being equivalent to a sand supply rate of 7,500 to 12,000 m^3 a year, which is assumed to apply at present. The range of uncertainty in the above figures is due to the irregular shape of the dunes and the method of applying a typical profile and photographic interpretation of limited accuracy to this.

As discussed separately later in this section, the sediment supply at West Beach has been influenced by the beach replenishment program. Between 1975 and 1981 approximately 150,000 m^3 of sand was placed in front of the dunes and approximately 120,000 m^3 at North Glenelg, 1.5 km to the south. This sand has partly protected the dune face from erosion, and would also have resulted in a decreased sand supply from the dunes. The beach replenishment sand is quickly spread by natural processes, and the effects on the dune face are indirect. It is not therefore possible to deduce, for example, the higher erosion rate that might have applied had the beach replenishment not been done. However, both the replenishment and the dune erosion contribute fully to the sediment budget.

At Minda. The Branch's estimate of annual dune face erosion between 1972 and 1981 is 1,000 to 1,500 m^3. This is based on an average dune face recession of 8 m during this period, over a length of 375 m. The sediment contribution from the West Beach and Minda dunes is likely to decrease if embayments form and if these retreat further behind the line of the adjacent seawalls. It will also be affected by beach replenishment programs and sea level rise. It is not possible to predict future rates of contribution from these dunes, though not unreasonable to assume that the rates will decrease.

There is evidence that these dunal areas can, under certain conditions, act as sediment traps. This was illustrated by recent severe beach loss at Somerton, immediately north of the Minda dunes. This loss appeared to be mainly due to the northerly drift of replenishment sand (placed further south at Brighton) being partly trapped and forming an accumulation in front of the dunes. There are two possible explanations for this. The first is that the wider beach fronting the dunes allows shoreward deflation and the accumulation of this sand into foredunes and beach berms beyond the reach of normal tides. However, the quantity would be small in relation to the alongshore drift, particularly considering that this phenomenon appears to be a short-term one. The other possible explanation is that these areas represent an interruption in the alongshore wave reflection and turbulence associated with the seawalls. A locally decreased rate of littoral transport could therefore be expected, with consequent sediment deposition. This would only apply for that portion of the tidal range when the sea was striking the seawalls, and would be influenced by the general levels of sand on the beaches.

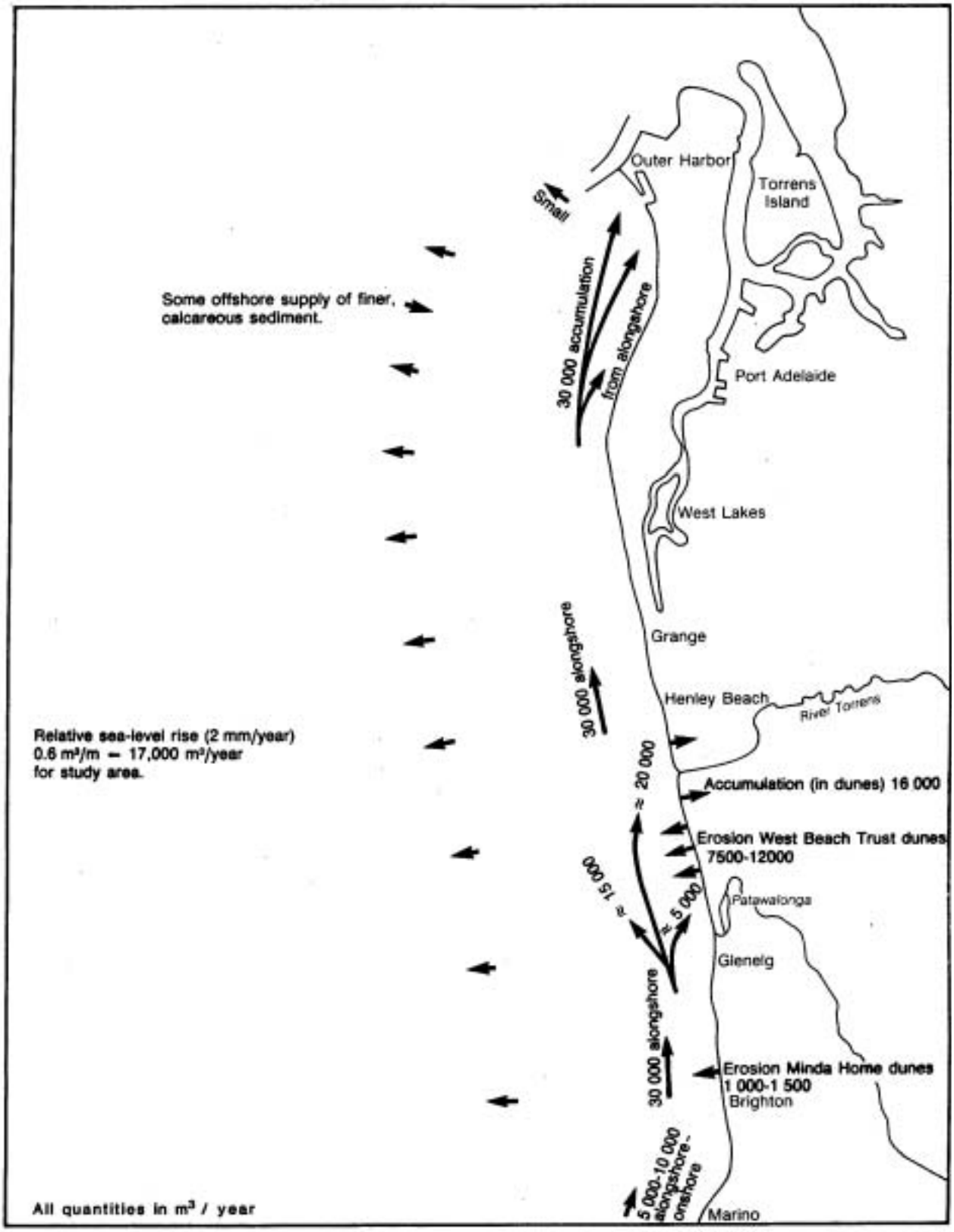
This anomalous behaviour at these dunal areas can be expected to be temporary. It may have local, short-term consequences, such as the beach loss at Somerton, but should not affect the overall sand budget.

In the event of major beach replenishment, both the seawalls and these dunes would be behind the new artificial dune and would cease being either a sediment source or a temporary sink.

Neither aerial photographs nor the beach profiles show continuing erosion of dunes between the Torrens Outlet and Semaphore. Storm erosion of these has occurred, but the dune face has been quickly restored by natural processes. In recent years losses in this northern part of the study area have been approximately balanced by gains, and the coastline position after the storm in 1981 was generally seaward of that after the major storm in 1953 (Moulds 1982). It is therefore assumed that there is no significant sand supply from dunes north of the Torrens Outlet.

The sand contribution from the two dunal areas is shown, together with other aspects of the sediment budget, in Figure 41.

FIGURE 41 AVERAGE ANNUAL SEDIMENT BUDGET WITHOUT BEACH REPLENISHMENT



Note:
1. Onshore movement is assumed small and in long term balance with offshore storm losses.
2. Possible offshore loss due to seagrass dieback is not shown.

e) Carbonate Production

The biological production of shell and coral fragments can be important sediment sources for beaches, though usually only in the tropics, where production is higher (Komar 1976). Although this would seem unlikely to be a significant source at Adelaide, changes in carbonate content along the beach suggest that it may be relevant to the sediment budget in the northern part of the study area.

There is a small gradual increase in carbonate content along the beach from Seacliff to Port Malcolm (refer to Section 2.2.3) and a marked increase to the north of Point Malcolm. The gradual increase could be partly explained by a more rapid alongshore movement of the lighter shell fragments, but this is unlikely to account for the high proportion of carbonates in the Semaphore and North Haven areas. It seems likely that there is an offshore supply of carbonate material to these northern beaches, but it is not possible to make an estimate of this.

Although the long-term quantities could be significant for these northern beaches, the sediment budget here has less relevance to the problems facing the beaches further south, which are the primary subject of this review. It has an indirect relevance because the greater part of the beach replenishment sand is obtained from north of Point Malcolm.

Because of the higher carbonate content, a portion of this replenishment material, possibly up to 10%, might be more mobile than the native sand on the replenishment beaches. The effect of this on the efficiency of the program would be slight in relation to other factors.

The present study addresses neither silting of the North Haven entrance nor its maintenance dredging, except insofar as this might be affected by the alternative protection methods considered (refer to Section 5.5.3). Any further detailed studies of the sediment balance for North Haven should take account of the apparent offshore supply of material with a high carbonate content.

f) Artificial Beach Replenishment

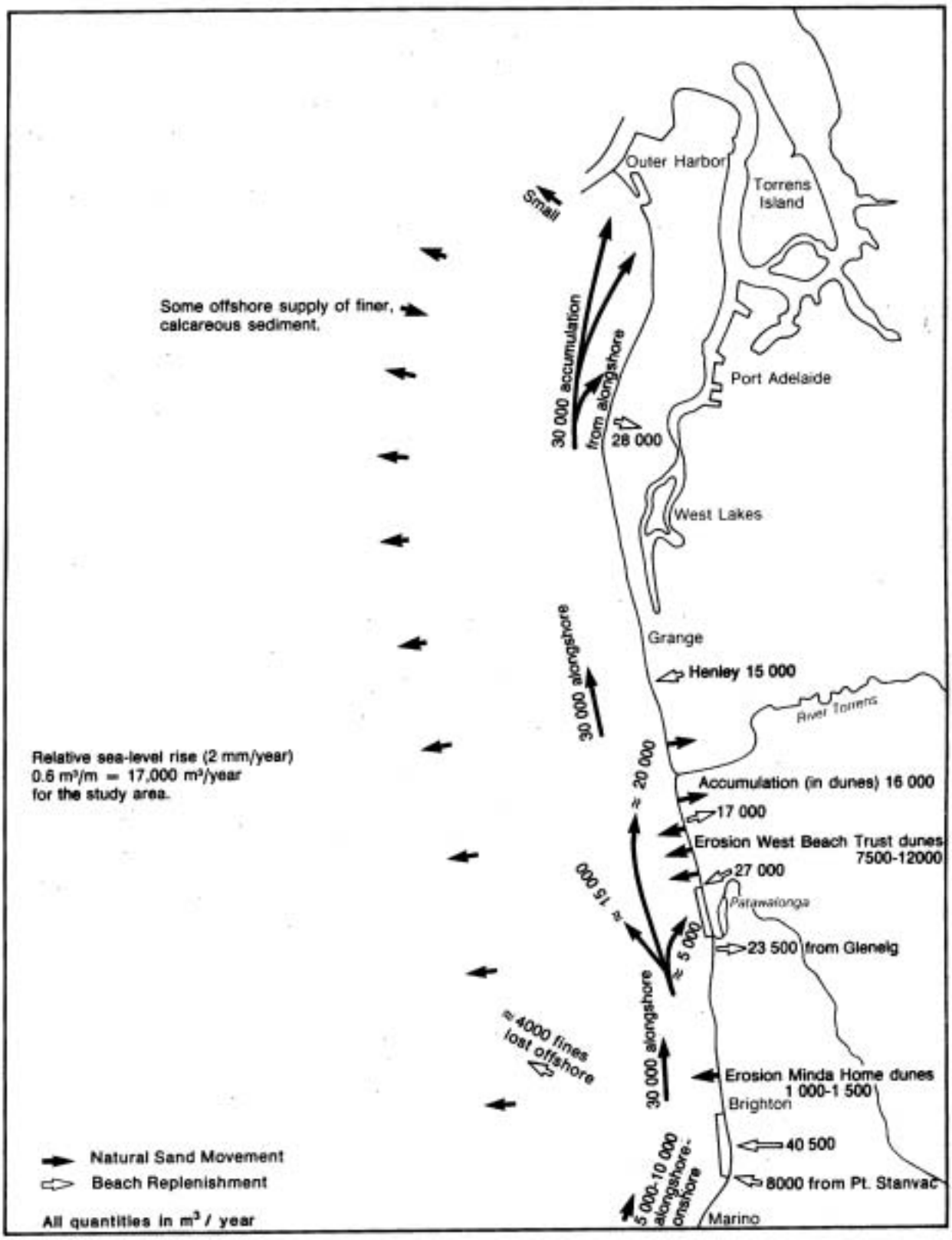
To avoid fragmentation of the information, both the removal and placement of sand is considered here, though the former is a loss from a locality and, to be strictly correct, should be dealt with under the 'Sediment Losses and Sinks' heading.

It is difficult to consider the effect of beach replenishment in the context of an annual, recurring sediment budget, and it has been necessary to make some assumptions. This is mainly because the program has varied over the ten years of its operation, and because some of the replenishment may still be in the process of natural distribution on the beaches. The most useful approach seems to be to assume the annual average for the main, recurring parts of the program, and to assume shorter-term averages for recently introduced parts of the program, such as the placing of sand at the new protection works at Henley Beach. The quantities given here and in Figure 42 are calculated on this basis. Actual replenishment quantities for each year are shown in Figure 34.

An adjustment is needed to take account of some of the introduced material not being compatible with the native beach sand. As discussed elsewhere in this report (Section 2.2.4), losses to offshore can be expected when the replenishment material is finer than the native material, or when the gradings are markedly different. Checks on the suitability of the replenishment sand have been conducted using the James (1975) method (CERC 1977). These show that overfill ratios in the range 1.7 to 4.0 may apply for material from Largs Bay, Taperoo and Port Stanvac (north beach), but that most of the sand has been compatible. (The overfill ratio is the ratio of the amount of sand placed to the amount remaining on the beach.) These three sources have been used for replenishment to the Brighton and Seacliff beaches only, and account for approximately 120,000 m^3 (less than a third) of the total to these beaches.

The calculation of replenishment losses is by no means reliable, and its local applicability cannot readily be tested. It is assumed here that a third of the 'suspect' replenishment sand is lost offshore. This represents a total figure of approximately 40,000 m^3/year. This is considerably less than would be obtained using the calculated overfill ratios, and further research is needed if any of the three 'suspect' sources are to be used in the future.

FIGURE 42 AVERAGE ANNUAL SEDIMENT BUDGET WITH BEACH REPLENISHMENT



Note:
1. Onshore movement is assumed small and in long term balance with offshore storm losses.
2. Possible offshore loss due to seagrass dieback is not shown.

Subject to the aforementioned assumptions, the significant annual contributions to the annual budget can be summarised in rounded figures as in Table 5. The number of years included in the averages is shown in brackets.

TABLE 5 AVERAGE ANNUAL BEACH REPLENISHMENT QUANTITIES

Sand Additions (m^3/yr)	Sand Removals (m^3/yr)
Brighton/Seacliff 40,000 – 4,000 losses = 36,500 (10)	Port Malcolm/Semaphore 28,000 (7)
North Glenelg/West Beach 27,000 (10)	Torrens Outlet 17,000 (9)
Henley Beach 15,000 (3)	South Glenelg 23,500 (10)

Sediment Losses and Sinks

This section considers complete losses from the study area (either through removal from the area, or through material remaining in the area but being permanently isolated from the beach processes), and losses from one part of the study area to another. Losses due to removal of sand as part of the beach replenishment program have already been considered and are not included.

a) Varying Alongshore Transport Rate and Effects of Structures

These two factors are related, and are conveniently considered together. Both, either together or separately, cause sediment to accumulate or to be lost from certain areas.

The most relevant aspects are: the availability of sand for transport, the offshore topography (as it affects refraction), the alignment of the coast (which influences the amount and direction of alongshore wave energy), and structures that impede or deflect sand offshore or that influence wave energy. Structures can influence wave energy by sheltering, by wave refraction and diffraction, and by creating extra turbulence and thus increasing alongshore transport (eg seawalls in the active beach zone). The state of equilibrium or disequilibrium of a beach can also be an important factor, which has special relevance to the spreading of beach replenishment sand.

Information on these critical aspects of the sediment budget is unfortunately limited, mainly because the alongshore transport rates are small in relation to local seasonal beach changes. This results in changes becoming measurable only after several years, and even then with little reliability. Measurement and understanding of the processes is made even more difficult by the changes themselves modifying the coastline and thus influencing further change. In addition, the extent of change may be constrained by physical factors. For example, an embayment between the ends of adjacent seawalls will form at a decreasing rate until an equilibrium shape is reached. Another example might be a groyne, such as that at Glenelg, where sand will first accumulate and then either be bypassed alongshore or deflected into deeper water. The computer model for calculating separate rates of alongshore transport for segments of the coast (refer to Section 2.5.4) shows promise as an aid to better understanding and prediction, but is still at a preliminary stage of development. At this stage, it cannot take account of non-linear effects.

The varying availability of sand for littoral transport is difficult to estimate. Obviously, for dune-backed beaches such as at Minda, West Beach and north of Grange, transport will not be restricted by lack of supply. The transport rate will be affected where there are seawalls, depending on the amount of beach fronting the seawalls and the tides reaching them – bearing in mind that much of the transport occurs during storms when tide levels are unusually high.

Where seawalls are combined with near surface clay or rock horizons (such as at Kingston Park and, to a lesser extent, between Somerton and The Broadway) transport rates will be lower. However, assuming that other factors are equal, the additional turbulence in these areas probably increases their transport potential. This seems likely to result in sand that enters these areas being rapidly transported through them. This effect is uncertain. Culver (1970) did model tests that showed an increased transport rate in front of seawalls, though this was limited by the consequent decreased availability of sand. This effect may apply at Brighton where replenishment sand south

of the jetty seems to be having less effect to the north than might have been expected. The potential transport rate fronting the rocks north of the jetty may exceed the rate where the sand has been dumped and where the sea does not reach the rocks. Sand entering the area north of the jetty from the dumpsite might thus be rapidly transported through it, with little or no improvement to the immediate beach. It should be noted that wave refraction effects also apply at the location, and that the real situation is more complex.

The variation in sand availability has been partly taken into account in trial runs of the University sediment transport model. A 'transport potential' factor has been included for each of 13 segments, though this single factor must allow for all influences on the transport rate, including that of impeding structures. Preliminary testing of the model has indicated areas of erosion and accumulation that accord fairly well with observed changes (Culver and Walker 1983). However, there are still some significant discrepancies. Of particular interest is the model's apparently accurate showing of accretion and erosion for the South Brighton beaches. Segment 10 (see Figure 40) shows as an area of accumulation, which could also help explain why sand placed there takes so long to spread and move northwards.

The offshore topography and the alignment of the beaches both affect the direction of the wave energy and consequently the rates of alongshore transport. The net northward transport rate will be highest for those parts of the coast most closely aligned to the predominant, south-westerly wave direction, and will be lower where the alignment most closely approaches a direction normal to this. Thus, the potential northward transport rate will be greatest at Marino Rocks, immediately north of The Broadway, and north of Point Malcolm. The lowest rates will apply at structures (eg the Glenelg groyne and the North Haven and Outer Harbor breakwaters) where the accumulated sand is aligned nearly normal to the resultant wave energy direction. Because of the fairly uniform offshore topography, the effect of coastal alignment exceeds that of wave refraction, except possibly in the Kingston Park/Seacliff area.

Neither the availability of sand nor variations in the transport rate can be quantified at this stage. These aspects of the budget are therefore not shown in Figure 2.38, but should not be ignored.

The state of beach equilibrium is a factor that has an important bearing on the alternative strategies considered in this review. It seems to be agreed (Silvester 1979 and pers. comm. 1983; Everts et al and R. Culver, pers. comm., 1983) that the rate of transport is lowest when the beach is at or near an equilibrium profile, and that it increases when the beach is out of equilibrium, whether this be due to a shortage of sand or to a surplus in one part of the beach system, as applies during beach replenishment. Silvester also suggests that the larger part of the alongshore transport takes place during the early stages of a storm, while sand is moving from the beach to the offshore bar to establish the temporary storm equilibrium, and during the latter stages when the sand is being returned to the beach by the flattening waves. He argues that the transport is influenced by the temporary disequilibrium.

Since most of the Brighton and South Glenelg beaches are short of sand, the alongshore transport out of the area may be higher than it would be if there were more sand on the beaches – though this assumes that there is sufficient sand to be moved, which does not apply at all places in the study area. This argument suggests that the alongshore transport rate might decrease after beach replenishment, once the new beach equilibrium had been established.

Beach replenishment, especially when on a large scale, upsets the beach equilibrium. A quite rapid adjustment may take place, with part of the sand being spread across the beach and nearshore to establish the new equilibrium. Alongshore transport may increase markedly during this adjustment, and may also be assisted in this by the alignment of the ends of the replenishment 'lump' being at a more oblique angle to the incoming wave energy, and also by refraction effects caused by the alteration of the beach and nearshore contours. Unfortunately these effects cannot be quantified and remain important unknowns. Reports from beach replenishment elsewhere confirm that the replenishment effect does spread along a coastline, though the time this takes seems to depend on the general level of wave energy and alongshore transport, and on whether the replenishment is spread over a length of beach or all placed in a small area to form a 'lump'.

An example is Atlantic City, USA, where it took several years for the sand to spread, despite a high energy wave environment (Evert et al 1974).

The spreading affects the alternative strategies in two main ways. Firstly, it cannot be assumed that replenishment of the southern beaches will cause a rapid improvement in those further north at Henley and West Lakes, or that it will cause any increase in width of beaches or alongshore transport as far north as Semaphore or North Haven. An improvement is probable north of the replenished areas, but the extent and the delay cannot be predicted. Secondly, this beneficial spreading could only occur through loss from the replenished area at rates higher than the alongshore transport, and this loss would need to be taken into account in deciding how much sand was needed. Again, no quantity estimate is possible.

The apparent early disappearance of some of the replenishment sand often causes public concern, to the extent that this can threaten replenishment projects. This concern is unwarranted, providing that the sand is coarse enough not to be lost too far offshore. Although not as visible, the sand will still be in the active zone where it is needed and, for the Adelaide situation, any sand that is 'lost' by accelerated transport or spreading northwards will be welcome at Henley Beach and West Lakes south, where it is also needed.

Structures influence the coast at three places relevant to this study. These structures are the groyne at Glenelg, the Torrens Outlet, and the southern breakwater of North Haven. The Outer Harbor breakwater has less relevance, because the northward moving sand is now trapped by the North Haven breakwater, and because the beach between North Haven and Outer Harbor can be considered to be isolated and effectively outside the study area. Estimates of rates of sand accumulation at structures are provided in Section 2.5.4.

Culver (1970) noted that the **Glenelg groyne** 'filled' within a few years of its construction. Also, recent average annual sand removal from south of the groyne at an average rate of 33 000 m^3 for the past 5 years has done little to reduce the sand accumulation or the blocking of the boating channel. This suggests that sand is passing the groyne in significant quantities, despite the sand removal. As illustrated in Figure 41, this sand is likely to move in three directions – offshore, along the bar to rejoin the coast further north, and onto the beach at North Glenelg. The erosion north of the Patawalonga suggests that very little sand comes ashore there. The persistence of this northward erosion indicates that, if sand does move ashore via the bar it does so considerably further to the north, probably near the Torrens Outlet. However, there is no firm evidence of this. There is likewise no indication of how much is lost offshore, though this seems likely to account for a substantial part. Significant volumes would certainly be moved offshore during flood flows through the outlet and during the low tide flushing, which is done in an attempt to maintain the channel.

Without sand removal from south of the groyne the quantity passing to the north and to deeper water could be expected to be greater than the estimated alongshore drift rate of 30,000 m^3 (refer to Section 2.5.4). This is because the groyne and the boating channel allow northward sand movement but limit southward movement, thus acting as a partial one-way valve. The actual rate would be somewhere between the estimated net rate of 30,000 m^3 and a possible gross rate of up to 70,000 or 80,000 m^3, probably nearer the former. Offshore losses may be associated with nearby seagrass loss, and could be linked with the losses considered here. These are discussed separately under the 'Offshore Losses' heading, and in Section 2.7.

Taking into account the sand removed for beach replenishment, the combined amount passing the groyne northwards may be approximately 10,000 to 20,000 m^3, most of which is probably lost to deeper water, from which it would return very slowly if at all.

There is no evidence to suggest that the remnants of the partly constructed breakwater off Glenelg significantly reduce wave energy or affect local beach processes. Any effect would have been noticeable as a sand accumulation before the groyne was built.

Sand has built up at the **Torrens Outlet** since it was cut through and the outlet structure built in 1936–37. The main build-up is south of the outlet, extending for a distance of approximately 900 m, and there is also a smaller accumulation immediately to the north, over a distance of approximately 300 m. The build-up is due to the flow from the outlet acting as a 'momentum groyne', which

partially interrupts the littoral drift along the coast. The outlet structure itself is behind the present dune line and does not interact with the coastal processes.

The behaviour of the outlet is typical of outlets on a coast with a predominant littoral drift. The channel changes its alignment across the beach in response to the changing directions of littoral movement, and mainly flows in a northerly direction, as would be expected. At times of strong stormwater flow, a more direct channel is cut to the sea. This helps in bypassing the sand moving alongshore, by releasing a 'slug' of sand. Full, or possibly reduced, bypassing occurs at low flow or between flows, with the groyne effect mainly occurring during significant flows. The channel is occasionally cut using earthmoving equipment to assist its bypassing and to avoid stagnant pools forming.

Culver (1970) noted that the sand accumulation was significant, but did not attempt to quantify it. An estimate has been attempted here, but the data is barely adequate, and the estimate has a low level of confidence. Estimates are based on the seaward movement of the dune face as measured from aerial photography by Moulds (1982), and on beach profiles surveyed since 1975. The average annual sand removal for beach replenishment of 15 000 m^3 a year for the past 10 years is taken into account. The main difficulty in using the dune face movements is that an assumption has to be made as to whether the whole coast has moved seawards, or whether the main accumulation has been above the water line, with the nearshore adjustment lagging.

The survey information indicates that the beach and dune accumulation has not been accompanied by a corresponding seaward movement of the beach profile, but rather that the nearshore slope has steepened, with a loss of sand, which, together with the sand removal, very nearly offsets the onshore build-up south of the outlet. Using position changes of the dune face between 1972 and 1981, and interpreting them in the light of the beach profile information, indicates only a small gain, of the order of 1,000 to 2,000 m^3 a year, for both sides of the outlet. If sand had not been removed for beach replenishment, the gain would have been of the order of 16,000 to 17,000 m^3 a year. A clear trend could be discerned from the survey profiles despite the considerable seasonal fluctuations, and the profiles are consistent with the aerial photography interruption.

The photography shows a large seaward movement of approximately 50 m between 1953 and 1981, and this would be more likely to have been accompanied by a seaward movement of the entire profile, despite this not having been noted for the shorter, more recent, period. Depending on the extent to which this seaward movement is assumed, the estimate average annual accumulation over the 28 years is in the range 15,000 to 23,000 m^3, allowing for the sand removal over the past 10 years. This is consistent with the estimate over the more recent period. An average annual accumulation of 17,000 m^3 is therefore assumed, though the quantity could be in the range 10,000 to 25,000 m^3.

It would appear that the rate of accumulation and of bypassing is not significantly affected by the local accumulation, and that sand can safely be removed from this area provided that the rate does not exceed the rate of accumulation. Although dune planting is necessary here to maintain a small buffer and to prevent sand drift nuisance, seaward dune building is unnecessary and should be discouraged as it merely locks up valuable sand. Northward dune building, on the other hand, is more useful as it will provide a small sand buffer seaward of existing rock protection.

The northern end of the study area is a 'sink', with the accumulated sand originally building the Le Fevre Peninsula and more recently building up against the Outer Harbor breakwater. Previous estimates of this accumulation, in the range 20,000 to 50,000 m^3/year, are discussed in Section 2.5.4. It was hoped that measurement of build-up against the new (1974) North Haven breakwater would enable a revised estimate. However, survey information has unfortunately proved inconclusive, indicating a net loss when the nearshore area is taken into account.

Volume changes were calculated using beach profiles measured between 1977 and 1982, and were compared with depth changes obtained from DMH soundings between 1975 and 1981. The most useful profile was that opposite Gedville Road, 850 m south of the breakwater. The other two profiles, at and adjacent the breakwater, were less useful because the former extends seaward

starting from the end of the breakwater and the latter is a 'wading' profile, which does not extend far enough to sea. The Gedville Road profile and that adjacent the breakwater confirm the visible beach growth, showing approximately 20 m^3/m of beach length/year accumulation averaged over the 6-year period. However, comparison of volumes above a horizontal datum at AHD –4.00 m shows neither an accumulation nor an erosion trend, and comparison of volumes above AHD –7.00 m shows a loss of sand at an average annual rate of approximately 60 m^3/m/year. A check on survey accuracy failed to identify this as a likely cause of the apparent discrepancy.

A comparison of the DMH soundings (May–June 1975, November–January 1979, and September 1981) confirms that a deepening is occurring beyond 600 to 700 m from shore. Trends are less certain at intermediate distances, and the Branch's interpretation of the surveys in the vicinity of the end of the southern breakwater differs from that of North Haven's consultants (Moffat and Nichol 1980). They measured rapid silting in this area, and predicted that this would accelerate.

The offshore deepening could be due to loss of seagrasses, probably caused by an initial shallowing. As discussed in Section 2.7, there is reason for reduced seabed levels following loss of seagrass, and there is evidence that this has occurred off West Beach. However, the direction of movement of the released sand remains unknown. It appears not to be moving ashore at North Haven. If it were, there would be an overall accumulation rather than a loss. A further complicating factor is the evidence of a sand supply to this area from offshore (finer sediment and higher carbonate content – refer to the previous discussion in this section and in Section 2.2.3). In the circumstances it is not possible to estimate alongshore transport from measurement of accretion at North Haven.

b) Losses to offshore

As discussed under 'Sediment Sources', the onshore/offshore part of the budget is indeterminate, though probably small, with a net gain being more likely than a loss. However, storms can move sediment seaward of the normal active beach zone and beyond the reach of the restorative fair-weather processes, causing losses from the system. This can be expected to apply especially when storms occur during an ebbing low tide and when there is a strong onshore wind. The onshore wind pushes surface water toward the coast, and a bottom seaward return flow is induced.

Research on this subject (reviewed in Komar 1976, King 1972 and CERC 1977) has so far been inconclusive. While it is becoming possible to estimate the overall storm effect on a beach, it is not yet possible to estimate the quantity lost to sea. The processes are complex, being influenced by the sediment parameters and beach slope as well as by the wave characteristics. Writers on the subject agree that the offshore loss is generally small in relation to other aspects of the budget. Komar (1976) notes that the greater part of any offshore loss occurs where sandspits project into deep water. This would hold true also for artificial sand projections such as at the Glenelg groyne, the effect of which has already been discussed – including the induced offshore loss.

Detection of the offshore movement is extremely difficult because very small level changes correspond to large volumes and precise measurement underwater is not easy. This involves the embedding of rods in the sea floor and the measurement of change against these. Although much better than lead or echo sounding, the accuracy is still poor in relation to the magnitude of the changes anticipated, and is influenced by changing seabed ripples. In June 1982 the Branch established a trial row of underwater rods near the Henley jetty, and has monitored these four times. No change in sand level has yet been noted. The rods are located 700 m south of the jetty extending from 300 to 500 m to sea.

Divers from the Fisheries Department and Engineering and Water Supply Department (Johnston, Shepherd, Steffenson and Olsen) and others have reported sand movements at wrecks and elsewhere offshore in both gulfs. The main observation at wrecks seems to be that build-up occurs on one side of the wreck and scouring on the other, but that there is not much other movement. This movement at wrecks has been observed at depths of up to 20 m. An exception is at the Tigress, which is in shallow water off Port Noarlunga. Much larger sand movements have been reported here, as would be expected, having regard to the water depth and the exposure to larger swell and storm waves. Sediment movement associated with seagrass loss and sewage outlet structures

off the Adelaide coast has been noted in water depths of up to 7 or 8 m, but generally not at depths in excess of 10 m. Ripples, which indicate some movement, have been observed on sandy bottoms at virtually all depths.

Komar (1976) also noted that offshore loss can occur through seaward diversion of the finer material in the alongshore drift, and that this could sometimes be detected by changes in the beach sand grading along the coastline. There is no local evidence of this occurring.

Culver (1970) suggested that greater seaward sand movement (with consequent greater offshore loss) would occur at seawalls and where there is a hard sea bottom, as occurs at Brighton and elsewhere in the study area. He confirmed by physical model experiment that sand was sent further out to sea in both cases. It is not possible to deduce from this whether or not a significant part of the loss was to seaward of the normal active zone, but it indicates that this is likely.

From the evidence available it must be concluded that there is some offshore storm loss from the budget, at least over shorter (5 to 10 year) periods, but that in the longer term this could well be balanced by gradual onshore movement of the same or other material. Such a balance is assumed for the purpose of the overall budget, and for figures 41 and 42.

The Alternatives Study assumed a 5,000 m^3/year offshore loss for the purpose of comparing protection alternatives. It is debatable whether or not this small loss occurs. However, it does not significantly affect the design or cost of the protection options.

c) Deflation Losses

As discussed in Section 2.5.2, deflation can account for a substantial inland sand loss and, as noted by Culver (1970), this has previously been a significant factor in the local sediment budget. However, sand drift control measures over the past decade have virtually eliminated this factor. The small continuing loss at the West Beach dunes has already been taken into account in determining the amount of sand that these dunes contribute to the budget.

d) Abrasion and Solution

Abrasion and solution of sand reduces the grain size, and it would be expected that this would result in offshore loss, due to the finer material being moved offshore by the beach sorting process. However, studies (reported in Komar 1976) have shown that particles smaller than about 0.25 mm do not undergo significant abrasion. It is thought that this may be due to the small particles having low inertia in collision with one another.

Since the mean grain size on the Adelaide beaches is less than 0.25 mm, abrasion is unlikely to be a significant cause of sediment loss. Beach grain size analysis does not show a marked reduction in grain size, except for the short length of beach between Semaphore and North Haven.

Studies reported in Komar indicate that the relatively soft calcium carbonate shell fraction is abraded much more rapidly, but that even this is surprisingly resistant when in very small fragments. Solution of the calcium carbonate component has been reported to be relatively slight. It can therefore be assumed that neither abrasion nor solution affect the short-term sediment budget or the coast protection alternatives considered in this review.

e) Inshore Loss by Human Removal

Beach sand is carried away inadvertently by beach users on their clothes and skin and in their shoes. It is also deliberately, though illegally, removed for use in bird cages, gardens and children's sandpits. Sainsbury (1974) estimated the total removal to be up to 1,000 tons (600 m^3 approximately) a year, a significant loss to be taken into account in the sand budget. The quantity is probably somewhat less than this, because Sainsbury appears to have overestimated the beach use, and because less material is now removed illegally. There is now greater public awareness of why sand should not be removed, and councils have been more vigilant.

The beach use analysis carried out by Kinhill Stearns and Riedel & Byrne as part of the Alternatives Study provides figures that suggest that the annual beach use for the study area is in the range 500,000 to 1,000,000 person-visits. This is an order of magnitude less than that assumed by Sainsbury.

It now seems unlikely that the annual quantity lost by human removal exceeds 100 m^3, and this only becomes significant over a large number of years. It is nevertheless important to remember that there is a critical shortage of beach sand, and that any sand deliberately taken will eventually need to be replaced at a public cost of approximately $5 per m^3 or more.

Effect of Mean Sea Level Change

A relative sea level rise causes the shoreline to move landwards but, strictly speaking, does not affect the sediment budget within the landward-moving system. The immediate consequence, as noted by Culver, is a loss of beach. The effect is in fact more complex than this, and needs to be considered in the sediment budget to enable beach replenishment quantities to be estimated. It has a marked effect on these.

Beach response to a rising sea level is not fully understood, and there remain conflicting theories. The prevalent theory (Komar 1976 and others) is that beach sediments are reworked and moved shorewards by rising sea level. This would tend to offset part of the sediment loss, when this loss is measured against a stationary shoreline. The other theory (Bruun 1962) is that the rise in water level is balanced by removal of sand from seafront dunes and redeposition of this on the seafloor to a thickness equal to the amount of sea level rise, and that this consequently results in landward movement of the beach. This is less consistent with current geomorphological opinion (Bird 1973, Pilkey and Evans 1982, Komar 1976, or Thorn, Bowman and Roy 1981), and is not supported by beach observation at places of known sea level change. It is virtually impossible to validate any theory on this because beach changes caused by seasonal changes in wave steepness obscure trends due to sea level changes.

Either theory requires that the subject beach is in a state of dynamic equilibrium, and this seems to be unlikely for most places. It seems more likely (Komar 1976 and Thorn, Bowman and Roy 1981) that, if equilibrium exists at all, it varies from place to place, depending on the energy regime and the supply of sand in the nearshore zone. Thorn, Bowman and Roy used radiometric and other evidence to examine the geomorphic and stratigraphic history of six coastal embayments in central New South Wales. One of their conclusions was that marine sediments had not been delivered to the shore during the late Holocene, though they noted that there was no evidence of relative sea level change during the period. Their conclusion does not necessarily apply to the present or to the study area, where there is evidence of significant relative sea level rise.

The preceding discussion assumes a natural, dune-backed beach with a sandy nearshore bottom. There is even greater uncertainty about sediment response to sea level rise where there are seawalls and other coastal structures, and where hard horizons in the nearshore zone are exposed or covered only by a thin layer of sediment.

In the absence of further information, the Alternatives Study used certain simplifying assumptions to obtain volumetric estimates needed for design of beach replenishment. The consultants assumed that a 2 mm a year sea level rise was analogous to scraping a layer of sand off the beach 2 mm thick over all of the active beach zone, which they had estimated to have an average width of 300 m. This gave an effective loss of 0.6 m^3/m beach length/year, ie approximately 17,000 m^3 for the coast between Seacliff and North Haven. The loss for a 10 mm sea level rise, which may prevail for short periods, would be at least 5 times this (approximately 85,000 m^3/year), though may be greater because the effect would extend further to sea.

The effect of a 2 mm rise was distributed on the assumption that the middle section of coast between Semaphore Park and West Beach was stable due to a balance between loss due to sea level rise and gain due to retention of some of the alongshore sediment transport. The consultants also assumed an alongshore transport of 30,000 m^3/year into the northern section, and ignored other effects such as intermediate contributions from dunes and losses offshore. Using this approach they deduced that the net gain to the northern part of the coast was 24,500 m^3/year (30,000 m^3 alongshore transport less 5,400 m^3 for the sea level rise for 9 km of coast). They argued that if the middle section was stable, the alongshore drift being retained must be 5,400 m^3/year (equal to the sea level loss for this 9 km of beach). They deduced that the net loss from the 8.5 km southern section must therefore be 40,000 m^3/year (35,000 m^3/year alongshore plus approximately

5,000 m^3/year for sea level rise). This approach is considered a useful one despite its dependence on the assumptions made and the uncertainties in these.

Lack of knowledge about either the sediment movement–sea level change process or the interaction of this with other aspects of the sediment budget makes it fruitless to pursue this at greater depth.

Relative sea level rise also has the indirect effect of accelerating cliff erosion and thus causing an increased supply of beach material. This would have an insignificant effect on the study area, because there is little transport of material from the south, where the cliffs are, and because the rate of cliff erosion is low. Also, as discussed, only the smaller, coarser fraction of the cliff material reaches the beaches.

Sediment Budget Conclusions

Despite the many uncertainties, it is possible to draw the following useful conclusions from this examination of the sediment budget.

1. The overall budget for the study area is close to being in balance if the effect of sea level rise is ignored, though the northern most part of the study area (at North Haven and Outer Harbor) may be an exception to this, as there is some evidence of a supply of finer sand from offshore. For most of the study area it seems probable that any offshore losses, as occur during extreme storms, are balanced in the longer term by a small shoreward movement of material from the nearshore zone. Local losses to offshore, such as at the Patawalonga groyne, are probably approximately balanced by the small supply of material from south of the study area.
2. There is a dominant northward littoral transport that results in a marked redistribution of beach sand within the study area, mainly by erosion of material from the southern beaches (at an average rate of approximately 30,000 m^3/year) and the deposition of this material in the northern part of the study area, where the alongshore transport is interrupted by the breakwaters at North Haven and Outer Harbor.
3. Coastal structures, eroding sand dunes and variations in the alongshore transport rate account for erosion at some places (eg West Beach and Minda dunes, and at North Glenelg) and accumulation at others (eg Torrens Outlet and Semaphore). Except for the beach replenishment program, which returns most of this sand to the active part of the system, much of the accumulated sand would be effectively lost from the system.
4. Relative sea level rise probably does not markedly effect the amount of sand within the beach system, but it does cause the whole system to move landwards. To counteract this effect (2 mm/year assumed) and to hold the coastline in its present position would require approximately 17,000 m^3/year of sand for the whole study area, and approximately 5,000 m^3/year for the southern, eroding portion. The effect of a 10 mm/year sea level rise would be more than proportionately greater.
5. The beach replenishment program has probably, over the past 10 years, more than offset the northerly drift (by approximately 10,000 m^3 a year). This possible slight margin is sufficient to offset the effect of relative sea level rise for the assumed design rate of rise of 2 mm/year, but would not be sufficient to counter the higher, short-term rises that have occurred in the past, and which may be occurring at present (see Section 2.3.4). Although critical to the protection strategies, this conclusion must remain speculative because the residual sand volumes are obtained by subtracting two items, neither of which has been conclusively established, and both of which are subject to large annual variation. If the conclusion is valid it is only so in the sense of being statistically probable. It is subject to both global and local climatic variations, as well as to other variable factors.
6. The present small-scale beach replenishment has complex interactions with the natural process at Brighton, and these appear to prevent an even northward distribution of the imported sand. Northward spreading of a major replenishment would be probable, with some consequent loss

from the replenished beaches. Neither the rate of this spreading nor the quantity can be predicted.

2.7 NEARSHORE SEAGRASS CHANGE

Seagrasses occur along most of the length of the study area in water depths between 2 and 18 m. They influence the coastal processes by reducing wave energy (through increasing bottom friction) and by trapping and binding sediment. Changes in seagrass occurrence, as have occurred to a significant extent in recent years, can thus be expected to affect the coastline.

This study has consequently included an assessment of seagrass changes and an attempted explanation for this. It also considers the possible implications that changes might be having on the coast and consequently on protection strategies.

Seagrasses play an important role in the local marine ecology by providing a habitat for bacteria and epiphytic growth, which, together with the detritus formed from dead leaves, is a valuable food source. Major loss of seagrasses, as seems to be occurring, could therefore have serious consequences for the local marine environment and fisheries production.

Explanations for the widespread recent losses of seagrasses need to be found, so that measures can be considered to prevent further loss, if such measures are possible or practical.

2.7.1 The Seagrasses

Only a brief description is given here. More detailed descriptions of the seagrasses and their occurrence are provided by Freeman (1982), Shepherd (1971), the Engineering and Water Supply Department (1975), and Shepherd and Sprigg (1976).

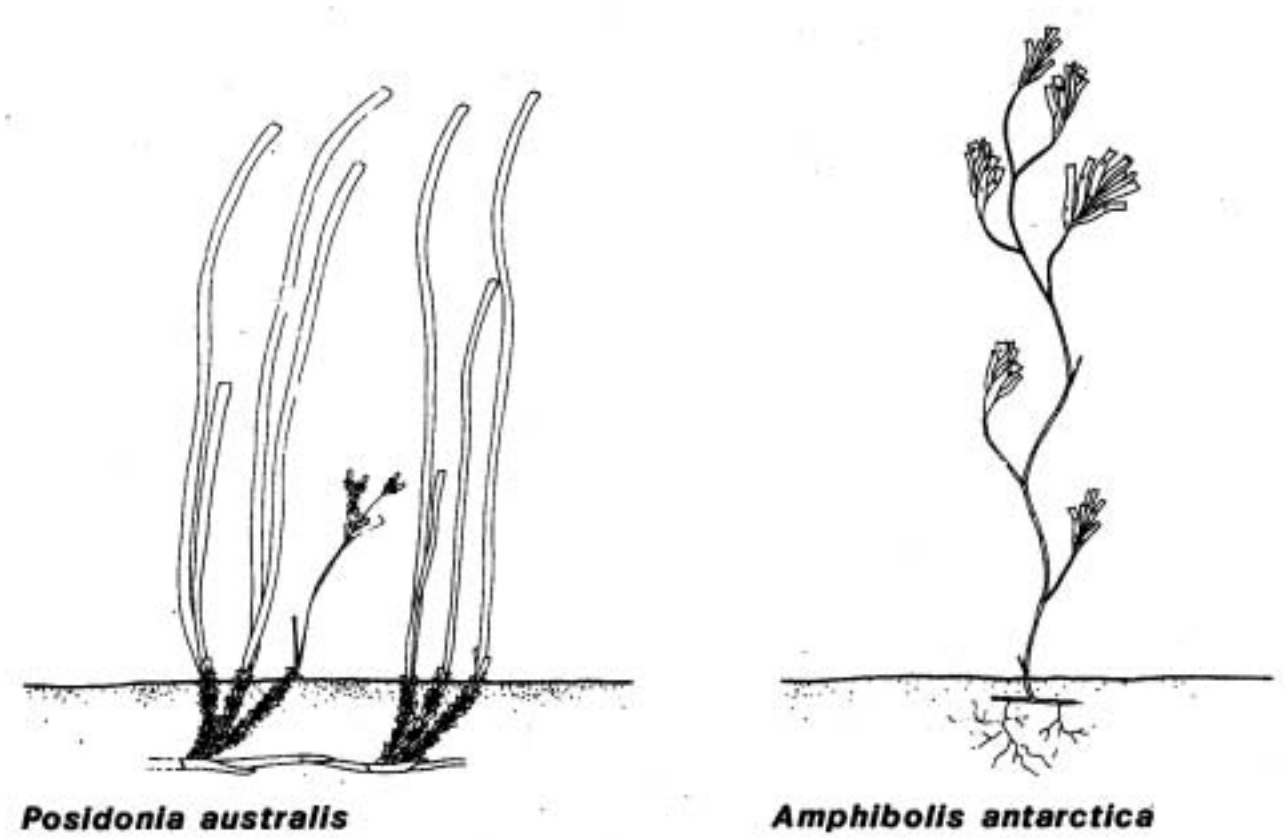
Several species of seagrasses are common in South Australian coastal waters, though only two, *Posidonia sinuosa* and *Amphibolis antarctica*, are dominant in the study area. It should be noted that *Posidonia sinuosa* has previously been described in the literature as a narrow-leaved variant of *Posidonia australis*, but that it has recently been identified as a separate species.

Both *Posidonia* and *Amphibolis* form mono-specific and mixed beds or meadows, which commence from within 100 m from the shoreline (usually from just outside the breaker zone) and extend seawards to the aforementioned depth of approximately 18 m – this occurring at distances of approximately 10 km from the shoreline. The landward limit seems to be determined by wave energy – where plants attempt to colonise water that is too shallow, they are uprooted by storm waves. The seagrasses thus colonise shallower water in places where the wave energy is least. An oversupply of sand, such as may occur in front of eroding dunes, may be a further factor limiting landward growth, because excess sand, together with sufficient storm wave energy, could contribute to smothering of the seagrasses.

Posidonia has ribbon-like blades, or leaves, of length up to 700 mm. These grow from roots and rhizomes, which form a dense system and which penetrate the sand to a depth of 600 mm. *Amphibolis*, which occupies the same and adjacent habitats, has tough stems with short leaves, and grows to a height of 400 mm. It has a network of cylindrical roots that penetrate to lesser depths of approximately 200 mm into the sand. The two species are illustrated in Figure 43.

Although neither species appears to be well adapted to colonising areas in which sand is mobile, *Amphibolis* appears to be able to withstand greater covering by loose sand. *Amphibolis* is considered to be better adapted than *Posidonia* to stronger water movements (Shepherd and Sprigg 1976). It has been found to be more capable of colonising areas where there are rocks and only shallow layers of sand, and is more abundant in the southern parts of the study area where these conditions are more prevalent. *Amphibolis*, if present in an area, is likely to attach and grow in the more stable areas before *Posidonia*, but other species may be even earlier colonisers of such sites (Fisheries Department comment on a draft of this report). *Amphibolis* is commonly found on the fringes of *Posidonia* meadows and to be colonising areas from which *Posidonia* has been lost.

FIGURE 43 THE DOMINANT SEAGRASS SPECIES



Shepherd (1971) suggested that *Posidonia* and *Amphibolis* may be associated in a natural cycle of succession, with the *Amphibolis* first colonising the rocky or mobile areas, stabilising these and trapping sediment, and then being displaced by *Posidonia*. It has been suggested (Engineering and Water Supply Department 1975) that when a *Posidonia* bed dies, the *Posidonia* is unable to stabilise it, because of the excessive sand mobility, and that *Amphibolis* is the more likely stabiliser in such events. A cyclic occurrence of the two species is thus possible. However, in commenting on a draft of this report, the Fisheries Department noted that when a *Posidonia* bed dies and the sediment becomes mobile, *Amphibolis* seedlings are probably less successful at colonising than *Posidonia*. *Posidonia angustifolia* was noted to be an early coloniser of blow-outs.

The relative occurrence of *Posidonia* and *Amphibolis* seems to change quite rapidly. The 1975 Engineering and Water Supply Department report noted that *Posidonia* had largely replaced *Amphibolis* in offshore metropolitan waters in recent years (1962–72), though this seems to be inconsistent with the regression trend, because replacement of the colonising *Amphibolis* by *Posidonia* suggests a trend towards stability rather than the reverse. During Engineering and Water Supply Department monitoring of the sludge outfall at Port Adelaide, 1976 to present, it was noted that quite large changes (as much as 30%) had occurred in stable beds, with the proportion of *Amphibolis* increasing (Steffenson, pers. comm., 1983). The change seems to be natural and perhaps cyclic, and cannot be quantified reliably or understood without extensive field study.

A natural feature in the growth of both species is the wide variety of epiphytic growth that attaches to their leaves and stems. The predominant epiphytes are encrusting red and filamentous brown algae (Engineering and Water Supply Department 1975). Many 'fleshy' red algae are also common on the thallus of *Amphibolis* (Fisheries Department comment on a draft of this report). The epiphytic cover increases as the seagrasses age, and could be a contributing factor to the death of the blades or possibly, as will be discussed, to widespread loss of plants.

2.7.2 Effect of Seagrasses on Coastal Dynamics

Seagrasses play an obvious and significant role in trapping sediment, and contribute towards the build-up of coastlines, such as in the upper Gulf St Vincent and Spencer Gulf, where coasts are building up by deposition of fine sediment. Shepherd and Sprigg (1976) report that studies in the upper Gulf St Vincent have shown that the seagrasses have caused the shoreline to migrate seaward several kilometres since sea level stabilised near its present level approximately 6,000 years ago.

The effect of seagrasses on the behaviour of sandy coasts with higher levels of wave energy is not as clear, and has not been subject to much research. Much of the information available has come from laboratory experiments on artificial seagrass, with limited verification in the field, and the findings have been contradictory. Discussion here, which includes discussion of these experiments, is based on the premise that the beach is in a state of approximate equilibrium between offshore storm loss and gradual shoreward movement of medium to coarse grain sediment. This assumption is discussed in the Sediment Budget section (2.6.3).

A laboratory and field experiment by Price et al (1968) indicated that the effect of artificial seagrass placed between the breaker zone and a nearshore sand supply was to promote landward movement of the offshore sand, with consequent improvement in the beach. However, their results can also be interpreted to argue that the artificial seaweed acted as a trap for the shoreward moving sediment, and at the same time reduced wave steepness, resulting in the formation of a 'summer' beach profile, with a reduction in the offshore bar due to sediment moving from it onto the beach. A further experiment, by Price and others (1970), indicated that the artificial seaweed did act as a trap, which actually inhibited landward sediment movement from offshore. They measured a loss of wave height and suggested that this was the reason for the improved beach.

These mechanisms are likely to apply to an even greater extent for natural seagrass in the Adelaide situation. This is because natural seagrass grows through the sediment it has trapped, building a progressively higher mound, which, together with the seagrass, would have a greater effect on wave height and steepness. The 'trap' effect would apply to both the offshore storm movements and also to any general onshore movement. Depending on the relative amounts of sand trapped, the seagrass could possibly contribute to erosion by trapping and preventing shoreward return of sand swept offshore during storms. This effect is probably smaller and less significant than the sheltering effect. It should be noted, however, that the sheltering effect is likely to give relief only from storm action and is unlikely to prevent longer-term sand loss or the effects of sea level change.

This explanation of the seagrass function can be extended to consider what might happen when seagrass dies off. The immediate effect would be that a large quantity of sand is out of equilibrium, having been held in a perched position by the seagrasses (Kinhill Stearns and Riedel & Byrne 1983). (The Branch's survey profiles at North Glenelg show clumps of seagrass on mounds between 0.5 and 1.0 m above the surrounding seabed from which seagrass has been lost.) The loss of seagrass could thus account for a large amount of sand becoming available to the beach or nearshore system. Quantities of the order of 3 to 5 million m^3 are obtained by applying the assumption that half of the total seagrass loss since 1949 is close enough to shore to have been at an average level of 0.5 m higher than it would be in the absence of seagrasses. This sand would be available to move either onshore or offshore, depending on its grain size and on the wave conditions at a particular time, though a net onshore movement would be expected.

In commenting on a draft of this report, Fisheries Department officers have questioned whether or not sediment levels have decreased as deduced from the residual 'high spots' on the profiles. Their diving observations suggest that sand is smothering present seagrasses at the same level and that the old mat is often buried – suggesting that sand levels might be increasing rather than decreasing in the seagrass degradation areas. They have also suggested that much of the material under the seagrasses is organic, and that this would influence the quantities of sand being considered. Conclusive comment on this is obviously not possible without further study.

Coastal progradation has been associated with seagrass loss at Geographe Bay, Western Australia (Paul and Searle 1978), and at Port Hacking, Botany Bay, New South Wales (M. Geary, Public Works Department, pers. comm., 1983). This argues against the suggestion that loss of seagrass has been a factor contributing to erosion of the North Glenelg coast. It is possible that it may be a mitigating factor, and that erosion, more probably due to the groyne effects of the Patawalonga structures and outflows and to the construction of seawalls, may have been even more severe had there not been concurrent loss of seagrasses. Notwithstanding this, individual storm events would have caused more single event erosion in the absence of the seagrasses than previously, because of the wave attenuation effects.

Shepherd (1971 and pers. comm. 1983) observed that moving sand was covering the former seagrass roots in places, up to depths of 100 mm. Although he explained this as being due to sand moving offshore from the beach system, it could also be explained in terms of sand being released from within the seagrass beds as the vegetation was lost from these. Without the binding and shelter from the seagrasses this sand would be above its normal, unvegetated equilibrium position, and could be expected to be highly mobile.

Whatever the mechanism for initial destabilisation, it seems probable that release of further large volumes of sand from within the seagrass beds is a contributory factor to further loss of the beds, and that the growth and destabilisation of seagrass beds plays a significant role in the coastal process.

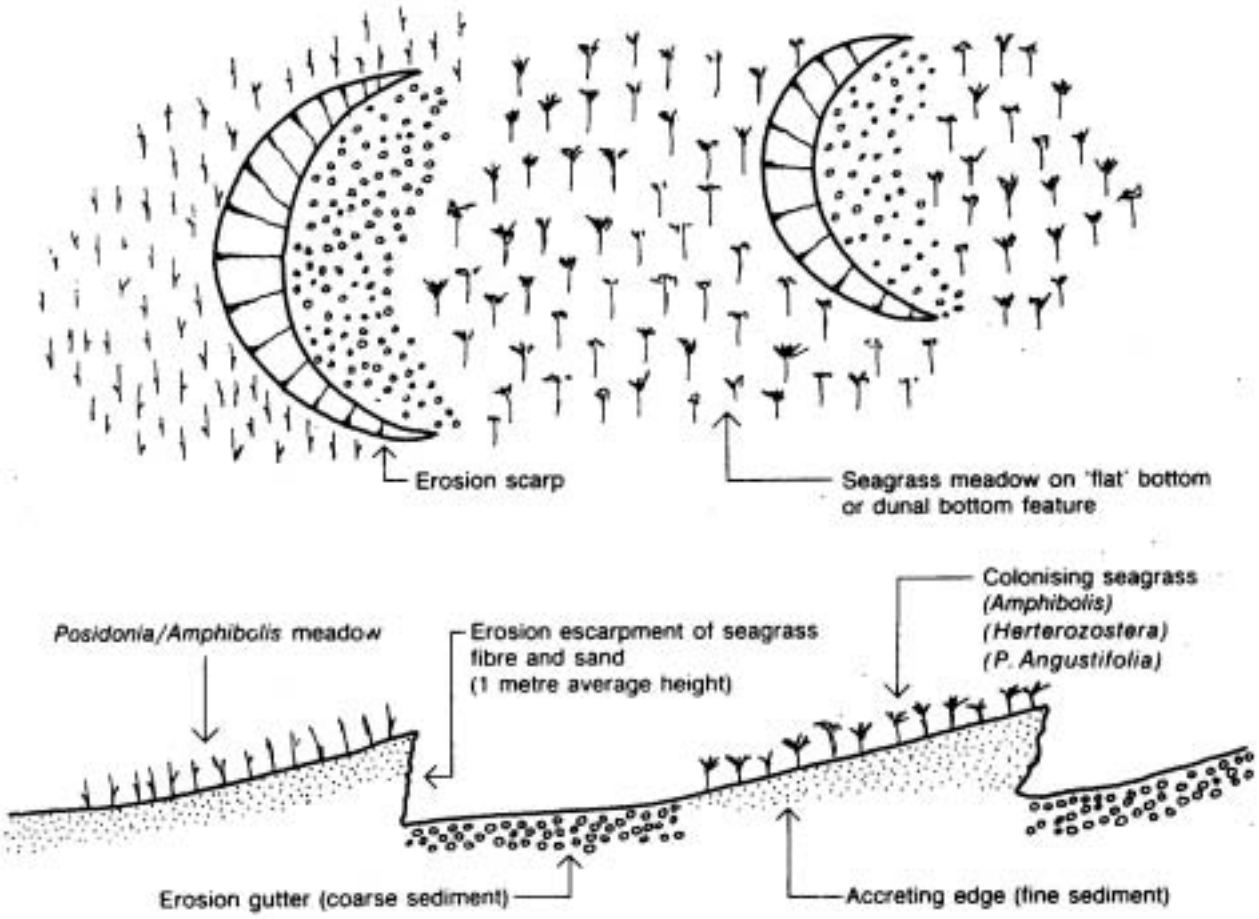
Cusp-like blow-outs are a feature of the nearshore seagrass beds that has been reported in all the aforementioned literature on the local situation. These blow-outs are analogous to terrestrial dune blow-outs caused by the wind, though it is uncertain how far the analogy holds, because of the superimposed oscillatory currents caused by the waves. Patriquin (1975) studied these in seagrass beds at Barbados and Carriacou, West Indies, and drew conclusions that seem to be locally applicable, though not perhaps entirely in agreement with the explanations put forward by Culver (1970), Shepherd (1971), the Engineering and Water Supply Department (1975), and Thomas (1983). Aspects of the explanations are, however, very similar. The following description is mainly based on Patriquin's findings.

The blow-outs appear to occur in nearshore seagrass beds where these have reached an upper level of equilibrium under the prevailing wave energy or sea current conditions. They seem to be a mechanism by which the equilibrium level is maintained by enabling seabed levels to be reduced when these have built up too high for the prevailing wave climate. At the same time, they provide an opportunity to preserve the overall health of the meadows through cyclic regeneration.

They have generally been related to disturbance by storm waves, though Scoffin (1970) reported their occurrence in response to tidal currents where these exceeded 0.9 m/s. A typical blow-out is shown diagrammatically in Figure 44. The Department of Fisheries has advised that blow-outs are known to occur in very deep water in the Mediterranean, where it is suggested that they are due to currents. Patriquin noted that the main axis and direction of movement was aligned with the predominant wave direction, and Scoffin noted that the axis was aligned to the dominant current direction. The eroding face, and in fact the whole feature, moves in a direction opposite to that of the waves or current, and where both apply, could be expected to move in a direction against the resultant of these two.

The normal process seems to be that a blow-out forms as a result of storm wave turbulence accentuated by some local irregularity on the seabed, or perhaps because of some weakness in the seagrass meadow or the sediment under it. This grows into the blow-out shape, which then progresses seawards through erosion of a steep, undercut seaward face and deposition of the erosion material on the flatter rear (landward) and side slopes. These flatter slopes are then colonised by the local colonising species (*Amphibolis* here), which is later replaced by the original succession species (*Posidonia* here). The sediment is reworked as the feature progresses across the seabed, with coarse material accumulating in the bottom of each hollow, and with this coarse layer subsequently being buried. The long-term effect could be that the area between the landward edge of the seagrasses and a seaward limit of wave influence might be continuously criss-crossed by these travelling features.

FIGURE 44 SEAGRASS BLOWOUT FEATURES



Thomas (1983) used 1949 and 1979 aerial photography to estimate local blow-out migration rates, and obtained figures of between 1 and 2 m of seaward movement a year. The higher rate was measured for water depths of 5.5 m and the lower rate for depths of 7 m. By comparison, Patriquin (1975) estimated a similar average rate of 1.3 m/year at Barbados, West Indies. No local estimate has been made of the frequency of passage of the features past a point, though the frequency would seem to be much less than Patriquin's estimate of 8 years for the Barbados seagrass beds. It should be noted that Thomas only examined a small area and that too much reliance cannot be placed on his estimates – bearing in mind the scale of the photography and the difficulty in horizontal control.

The blow-outs are reported by divers (Steffenson, pers. comm., 1983) to be more common and to be larger in deeper sediments than where shallow sediments overlie a hard horizon. In the event that these features are connected with the seagrass regression trend, this could offer some explanation for regression occurring in the Glenelg to Henley area, where sediments are deeper, and not in Brighton, where they are generally shallower.

None of the authors (except Thomas 1983) addressed the question of the direction of mass transfer of sediment during passage of the blow-out feature. Thomas postulated that there was a seaward movement, ie in the same direction as the movement of the blow-out. He explained this as being due to vortices (formed by interaction of the eroding face of the blow-out with the oscillatory bottom currents induced by storm waves) being moved seaward by a net seaward movement of the water near the seabed, which he assumed to occur during storms, and thus carrying some suspended sediment seawards. This hypothesis remains to be tested. Current measurement within a blow-out forms part of a detailed study of the local seagrasses that will be starting in early 1984.

Although this explanation may be correct for a part of the sediment, if applied to the total mass transport, it would seem to be at odds with almost all other observation of movement of fluvial material subject to wind, current, or other forcing mechanism. Blow-out type features in particular tend to move against the direction of wind or current, with the mass transport being in the direction of the wind or current (cf behaviour of blow-outs in sand dunes, and Scoffin's (1970) observations of current-induced seagrass blow-outs). It is possible that the superimposed wave effect could reverse the direction of movement of the feature, but this seems unlikely, and has not been proved. The seaward movement of the blow-outs would seem to suggest that the mass transfer is landwards, which is what would be expected as a general trend (see discussion in Section 2.6.3). However, seaward sediment movement might be expected during storms and, if it is assumed that the blow-outs are primarily a storm feature (which seems to be agreed), then the blow-outs should migrate in a landward direction – ie against the direction of the net seaward bottom currents assumed to occur during storms. Since the blow-outs do not migrate landward, either seaward storm sand movement does not occur at these depths or, if it does, is not associated with the blow-out features.

Paul and Searle (1978) investigated the behaviour of shore-transverse ribbon trenches through seagrass beds in Geographe Bay, Western Australia. They concluded that the trenches, which were aligned in a north-west–south-east direction, were affected by both ocean swell (the orthogonals of which were parallel to the trenches) and locally generated sea waves, with orthogonal direction approximately at right angles to the trenches. The features moved against the direction of the locally generated waves, with a leading scarp (similar to the blow-out features) and with sand being removed from the scarp and being deposited on the sloping trailing edge. This edge was being colonised with *Amphibolis*. The authors did not indicate whether there was a net mass transport in either direction, but their description suggests that, if there was, it was in the wave direction. The investigation was more concerned with shoreward sediment movement along these furrows, this movement being driven by the swell, and with coastline change associated with this.

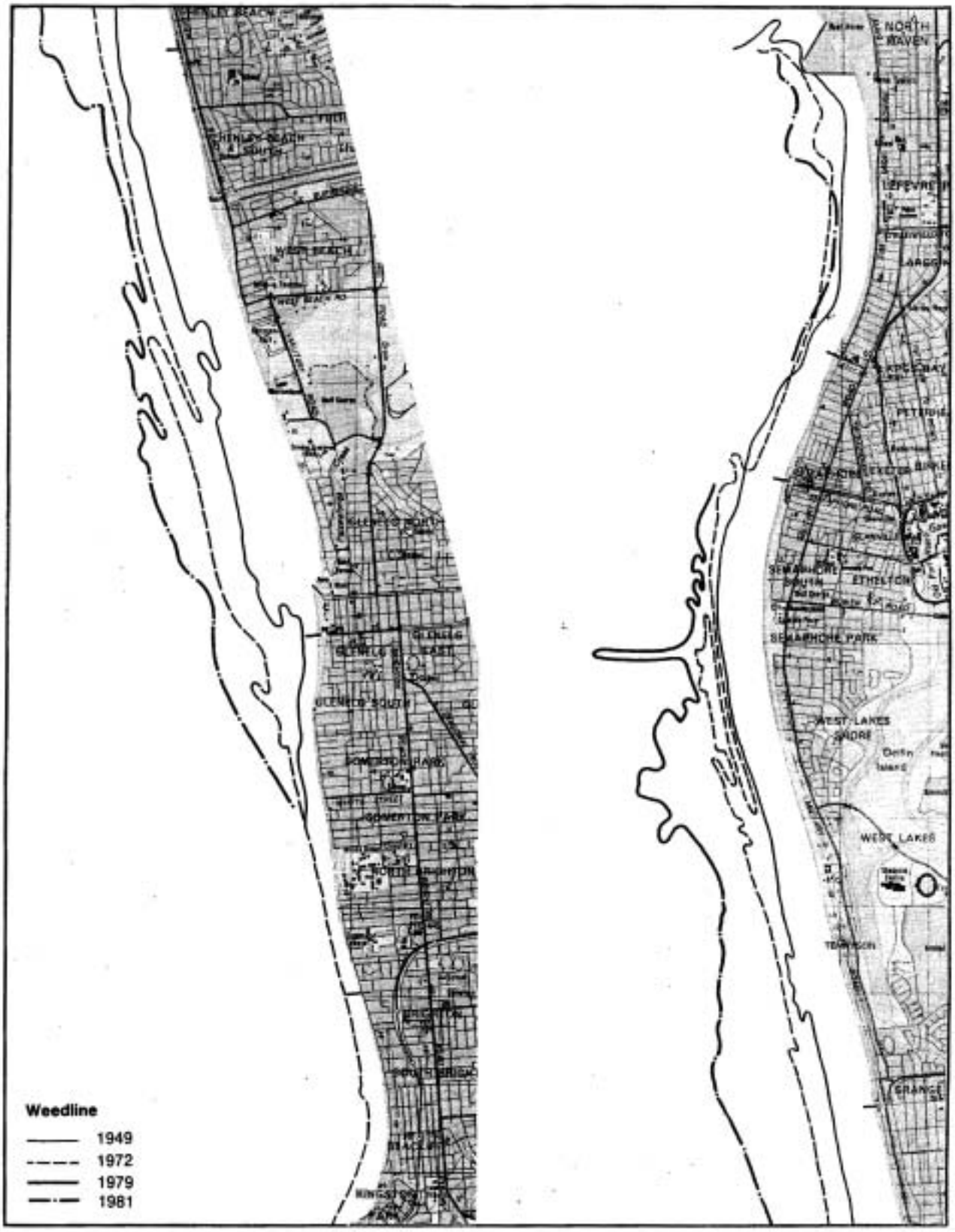
On balance, it seems likely that movement of the local blow-outs is not strongly related to offshore movement of sand during storms, and that offshore sand movement most probably occurs mainly inshore of the seagrass beds. Mass transport associated with the blow-outs may well be very small and, subject to further study, would seem most likely to be in a shoreward direction.

2.7.3 Measured Changes

Die-back of seagrasses seems to have been recognised as an issue of importance since Shepherd's work for the Department of Fisheries between 1968 and 1972. The Engineering and Water Supply Department addressed the subject as a major issue in its report, Gulf St Vincent Water Pollution Studies 1972–75, and included in this a mapping of seagrass regression. The report does not state the period during which the mapped regression occurred, but mentions that aerial photography dated 1949, 1956 and 1971 was available, so it presumably covers these years. The regression was not quantified.

Culver (1970) plotted the shoreward boundary of the seagrasses from the 1949, 1969 and 1970 aerial photography and concluded that there was very little variation over the period of this photography. He placed greater emphasis on 'erosion trenches' (the 'blow-outs' already discussed) within the main inshore body of seagrasses. These had been noticed on the photography as inexplicable low points on survey profiles and by diver inspection. He followed the view of Shepherd (1971) that these trenches were erosional features caused by storm waves scouring the *Posidonia* and its root mats. He suggested that the subsequent filling of these with sand from the beach could explain an offshore sand loss not being detected by survey. Having regard to the previous discussion, this seems unlikely. Culver noted the presence of these features both within the area that subsequently regressed rapidly (between 1970 and 1975) and further south, off Brighton, where regression has not occurred.

FIGURE 45 SEAGRASS RECESSION, 1949–82



Because of reports that the regression might be worsening, and because of the perceived relevance to the Strategy Review Study, another, more detailed, assessment was made by the Coastal Management Branch (Freeman 1982). Reference here will be mainly to this report. The Engineering and Water Supply Department has recently carried out further biological and chemical studies, but the report on these is not available at the time of writing. Discussion with the officers involved suggests that the findings are unlikely to contradict the discussion here.

Freeman mapped the shoreward boundary of the seagrasses using 1935, 1949, 1961, 1972 and 1981 aerial photography. He also mapped the 1975 position for areas where large-scale regression had occurred between 1972 and 1975, to enable a better assessment of recent variations in the rate of loss. In parts of the coastline where 1981 photography was not available, 1979 cover was substituted (see Figure 45).

This study showed a total regression of approximately 1360 hectares between Kingston Park and Outer Harbor for the period 1949 to 1981. Gaps in the earlier photography prevented overall quantifying of change since these earlier years, but these were measured for those areas covered. It was found that, for the whole study area, nearly two-thirds of the regression had occurred since 1971.

Two main areas of regression were noted – between Whyte Street, North Brighton, and West Beach Road, and in the Taperoo–North Haven–Outer Harbor area. The mapping for these two areas is reproduced in Figure 45. There was almost no change between Brighton and Somerton Park, and a steady, but slow regression in other areas. The results of the study are summarised in Table 6.

TABLE 6 SEAGRASS RECESSION 1935–1981 (FREEMAN 1982)

Zone	Period	Area of Die Back (ha)	Percentage of Study Area	of Zone	Average Rate (ha/yr)
Total Study Area	1949–81	1,360	100	100	42.5
	1949–72	520	38	38	22.6
	1972–81	835	62	62	92.8
Whyte Street to West Beach Road	1949–81	470	35	–	14.7
	1949–72	240	18	–	10.4
	1972–81	230	17	38	25.6
	1935–81	605	–	100	13.1
	1935–49	135	–	22.5	9.6
	1949–61	60	–	9.5	5.0
	1961–72	180	–	30	16.4
Henley Beach Road to Point Malcolm	1949–81	690	51	–	21.6
	1949–72	175	13	–	7.6
	1972–81	515	38	56	57.2
	1935–81*	925	–	100	20.1
	1935–49*	260	–	28	18.6
	1949–61*	110	–	12	9.2
	1961–72	40	–	4	3.6

* The 1935 and 1961 lines have been interpolated.

Note: Only the relevant percentages have been calculated.

The summary of the results was:

- 86% of the dieback that has occurred in the period 1949 to 1981 has occurred in the areas between Whyte Street and West Beach Road and between Henley Beach Road and Point Malcolm.
- 94% of the dieback in these areas occurred between 1972 and 1981, the major proportion of it between 1972 and 1975.
- These areas, except from Audrey Street to Cormorant Court, have been relatively stable between 1975 and 1981.
- A similar regression trend to that which has occurred south of the Outer Harbor revetment is becoming apparent in the vicinity of the North Haven revetment.

2.7.4 Causes of the Seagrass Recession

Reasons for the observed recession remain uncertain despite much conjecture in recent years.

Shepherd (1971) suggested that nutrient discharges (from the Glenelg Sewage Treatment Works) could be stimulating the epiphytic growth on the seagrass blades. This growth reduces light reaching the blades and affects respiration, progressively reducing growth of the blades, and eventually causing them to fall over under the weight of the epiphytes. He also noted that seagrasses were being smothered by highly mobile sand, which he considered a secondary effect, resulting from the initial loss of the vegetation. Kinhill Stearns and Riedel & Byrne (1983) supported this general explanation in the Alternatives Study, and suggested that storm wave erosion of the seagrass beds was another possible cause of initial seagrass loss as well as being a secondary factor causing further loss once the sand had been destabilised. They showed that the nitrogen supply from the treatment works (estimated 676 x 10^3 Kg TKN/year) was far higher than that from the Torrens and the Patawalonga, and that the load from the Torrens (estimated 93 x 10^3 Kg TKN/year) was considerably greater than that from the Patawalonga (0.06 x 10^3 Kg TKN/year).

The Engineering and Water Supply Department (1975) did not support the nutrient theory, and suggested that excessive sand movement, resulting from structural modifications to the coastline, was the main cause. Regression was attributed to both loss of sand in the Brighton and South Glenelg areas and accumulation of sand at Largs Bay. The Engineering and Water Supply Department stated that there was no evidence to support the nutrient theory, claiming that nutrient levels from the Treatment Works had never approached those reported as promoting eutrophication in other situations, and that no evidence of eutrophic affects had been found along the metropolitan coastline. The Department reported that a substantial epiphytic cover of inshore seagrasses was a natural occurrence throughout the metropolitan area and those other areas used for control in their study. Attention was drawn to the low nutrient levels in the Largs Bay area, where regression was also occurring, and to the likelihood that an excess of nutrients would shorten the life spans of seagrasses, and cause shorter growth cycles rather than result in total loss.

Discussion here excludes the regression at Largs Bay, because this is almost certainly due to seaward progradation of the beach, with a corresponding retreat of the seagrasses. The loss at Largs Bay may have been partly due to smothering by the incoming sand, or by the grasses growing through this new sediment and then being scoured out from this shallower water by storm waves. The latter explanation seems more likely, and is more consistent with the unusually high amount of dead seagrass that is washed ashore in this general area.

It is important to note here that no new work has been carried out in this study other than the photo-interpretation described and a small amount of diving and bottom sampling done by Thomas of Kinhill Stearns. It is therefore not possible to throw much new light on the subject other than perhaps to incorporate the foregoing observations about the seagrass bedform changes and the relation of these to onshore/offshore sediment movement.

Before considering the likely causes in more detail, it is worth providing brief notes on possible factors that have been discarded for one reason or another.

Factors Considered Unlikely to be Causing Regression

Although the possible factors are discussed separately in the following, it should be borne in mind that several factors could apply together. For example, turbidity, epiphyte abundance and sediment adhesion all reduce light and could have a combined effect.

a) Water Turbidity

Water turbidity reduces light penetration and could therefore reduce photosynthetic activity in seagrasses. It must be considered as a possible factor because turbidity has undoubtedly increased since construction of the Torrens and Patawalonga outlets and with the discharge of effluent off Glenelg. The only previous discharge along this part of the coast was through the original natural outlet of the Patawalonga, and this was ephemeral.

The Engineering and Water Supply Department (1975) discounted turbidity as a cause of seagrass regression, mainly because sampled turbidity levels were relatively low, because they were lowest in Largs Bay where seagrass regression was also occurring, and because a control area at Normanville showed no adverse effects despite considerable turbidity from three local streams.

Kinhill Stearns and Riedel & Byrne noted that the total annual amount of light received by the shallower regressing seagrass beds (at 5 m water depth) probably exceeded that received by the extreme seaward beds (at 18 m water depth), allowing for the temporary periods of higher turbidity in nearshore waters. They noted that the Torrens Outlet was a large, relatively new discharge, and recommended that turbidity not be ignored, and that it warranted further investigation.

Initial loss of seagrasses, perhaps by some other mechanism, could be expected to result in increased turbidity by re-suspension of fine material that had settled through the seagrasses and been stabilised by them. Although documented elsewhere (Davies 1970), this effect is unlikely to be significant in the local situation, where sediments in the regressing seagrass beds are relatively coarse in comparison to those that would be expected in areas of lower wave energy or in deeper water.

Notwithstanding the local view that turbidity is not a probable factor, Larkum (1976) considered that it was a cause of seagrass loss in Botany Bay, New South Wales.

b) Salinity Reduction

The Engineering and Water Supply Department (1975) measured salinity and found that water of low salinity (discharged from either the sewage treatment works or the stormwater outlets) tended first to form a thin layer on top of the seawater and then to disperse rapidly by wave and tidal action and through diffusion. It was concluded that salinity had negligible effects on seagrass communities along the metropolitan coastline. In addition, seagrasses grow in water with a wide salinity range and in places where salinity is low after periods of heavy rainfall (Den Hartog 1970).

c) Disease

No evidence of either fungal or viral disease has been reported, though this possibility should not be totally dismissed. Kinhill Stearns and Riedel & Byrne note that *Zostera* meadows in North America and in Holland have been affected by ‘wasting disease’. They note also that fungal infections are usually observable.

d) Temperature Change

Temperature change has been known to affect seagrasses, which do have temperature tolerance limits (Thayer et al 1975), and Rasmussen (1975) describes massive seagrass loss off the North Atlantic coast in the early 1930s as being due to a fungus infection that followed a rise in temperature of the Gulf Stream. Rasmussen (1975) postulated that the temperature rise not only lowered the resistance of the *Zostera* to infection but also made conditions ideal for fungus to multiply.

Steffenson (reported in Freeman 1982) considered that outfalls into the gulf would not alter its temperature range of 15°C to 23°C, which was well within the tolerance limits of the locally occurring seagrasses.

e) Toxic Pollutants

The Engineering and Water Supply Department Report (1975) notes that various pesticides, fertilisers and industrial chemicals are discharged into the gulf waters, but notes that dispersal studies show that it is most unlikely that these agents could be associated with seagrass degradation. The report notes also that no comparable studies have produced evidence that seagrasses can accumulate or be adversely affected by trace amounts of these chemicals. Harlin (1975) has suggested that nitrates may be toxic to seagrasses, and Thomas (pers. comm., 1983) has proposed that the lack of nearshore seagrasses at Wallaroo may be due to a former discharge of soluble copper and to continued leaching of copper sulphate from the remaining coastal slag heaps. A study of the marine effects of lead and other metals from the Port Pirie smelter (Ward et al 1983) found that seagrass growing in the contaminated area was less productive but appeared healthy despite high metal concentrations.

Local evidence on the effect of pollutants is clearly insufficient to discount the possibility that they may be causing seagrass loss.

The Probable Causes

Two assumptions are made here. First, since smaller seagrass recession is not known to be occurring elsewhere adjacent to the South Australian coast (other than possibly at Wallaroo), and since the recession commenced in recent times, it is assumed that seagrass recession is related to development by man – most probably to nearshore discharges or to another of these factors that initiated the regression, rather than a combined effect. There is a possibility that entirely natural events, such as a possible long-term cyclic fluctuation about a nearshore equilibrium seagrass growth or a change in wave climate, could have initiated the regression. However, the coincidence would seem to be unlikely, and there is no evidence to support either possibility.

Once the seagrass started to decline, the large amount of sand released would undoubtedly have been a major factor (and probably the dominant factor) in further recession. This presents a difficulty, because the original causative agent may no longer be present, or may be obscured by the more obvious present processes. Separation of the effects may not be possible.

a) Excess Sedimentation and/or Scouring

The possible mechanisms for seagrass recession due to sand movements caused by coastal structures (or possibly due to sand mobilised for some other reason) are:

1. Coastal structures do cause sand to move further offshore during storms and generally increase the mobility of sand on the beaches and in the nearshore zone (refer to Section 2.6). Groynes such as that at Glenelg deflect sand into deeper water, and large stormwater discharges also sweep littoral material offshore, possibly into the seagrass beds.
2. The structures cause updrift erosion. Sand available from eroded dunes may be temporarily available to be moved offshore during storms, onto the seagrasses.
3. Adjustment of the nearshore bathometry to suit a retreating coastline is associated with a deepening in the vicinity of the nearshore seagrasses, which consequently become out of equilibrium. Initial adjustment is effected through the scour effects of storm waves.
4. Sand released by this process becomes available to smother adjacent seagrasses.

This explanation appears, at first glance, to be likely, especially because the first seawalls were built on the South and North Glenelg coasts in 1860 and 1925 respectively – early enough for them to have caused the initial seagrass loss. It also ties in with observations that sand movement seems to be the main cause at present.

However, the following observations argue against this explanation:

- The Patawalonga Outlet and groyne, which are likely to have had the greatest effect, were constructed in 1964, well after seagrass regression started.

- There has been no regression at Brighton, where there is the most interaction between the sea and the seawalls. This may be partly explained by the shallower nearshore sand layers, and because of the relatively higher proportion of *Amphibolis*, which is more able to tolerate mobile sand. A further consideration is that Brighton, being at the upstream end of the littoral drift, has a more regular bar formation, which is not as liable to onshore/offshore position fluctuations as the bars further north.
- Recent evidence, from the tracer experiments and beach profiling, suggests that storms do not generally move sand as far seaward as the seagrasses, even at Brighton, where the effect could be expected to be greatest because of wave reflection from the seawalls. Sand movement can occur, and obviously is occurring, in the nearshore beds, but may be a consequence of the release of sand within the beds rather than a transfer of material from the beach.
- The sedimentation/scour mechanism fails to explain why there has been no re-colonisation. It should be noted, though, that the epiphyte growth theory also does not provide an adequate explanation for this. It seems more likely that re-colonisation is prevented by the increased sand movement associated with the loss rather than by whatever factor originally caused the regression to start.

A seabed elevation was noted by Hails and others (1983) and by Kinhill Stearns and Riedel & Byrne (1983) in the seagrasses immediately seaward of the regression area off West Beach. Although it was argued in the Alternatives Study that this is indicative of an offshore sand movement, it could also be explained as being due to the grasses trapping part of a to-and-fro movement, which may well have a net shoreward resultant – this net shoreward movement has already been argued to be more likely than an offshore movement (Section 2.6).

If the present loss is due to storm wave–induced movement of released sediment, then a seaward limit could be expected, as the bottom effect of storm waves becomes insufficient to move sand at a greater rate than it can be recolonised. A marked decrease in the regression rate since the 1972–75 peak could be due to this effect.

b) Excess Epiphytic Growth Due to Nutrient Discharge (mainly from the Glenelg Sewage Treatment Works)

As already noted, this possible cause of the seagrass recession was first mooted by Shepherd (1971), was argued against by the Engineering and Water Supply Department (1975), and was again suggested as the most probable cause in the Alternatives Study. The likely mechanism is as follows:

1. Epiphytic growth on the seagrass blades is accelerated due to a higher than normal level of plant-available nitrogen, both in the seawater and absorbed by the seagrasses. The excessive epiphytic growth reduces photosynthetic activity (and possibly respiration), and contributes to physical damage to the leaves through its weight and through the extra water resistance that it causes.
2. The effectiveness of the vegetation cover is thus reduced and storm waves can more readily scour out the partially bare patches. The weakened seagrass bed may also be less resistant to smothering by sand movement onto it.
3. Recolonising grasses may be weakened by early epiphytic growth, or may be unable to survive in the highly mobile sand released from adjacent regressing beds.

The following arguments support this explanation:

- Although quantities were probably small, septic tank discharge commenced quite early (1904), and later seepage would have occurred from sewage disposal behind the North Glenelg dunes. The first major outfall pipe was built in 1943, which considerably pre-dates the construction of the Patawalonga groyne, and which appears to correlate reasonably well with early seagrass loss (see Table 6).

- There is strong evidence of a high level of epiphytic growth and that this has been responsible for seagrass dieback elsewhere, and also that such growth can be due to enrichment of waters by nitrates (Alternatives Study).
- Tidal current excursions from the effluent outlets during calm conditions are consistent with the affected area (based on the interpretation by Kinhill etc of dye tracing done by the Engineering and Water Supply Department in 1974, and subject to the accuracy of this method, which shows surface water movements rather than those at the seabed).

Arguments against the epiphyte explanation are:

- Observations by the Engineering and Water Supply Department in 1974–75 are reported as showing no correlation of epiphytic growth with seagrass die-back (Engineering and Water Supply Department 1975).
- The Engineering and Water Supply Department report states that the study had not found nutrient levels approaching those that had been reported as promoting eutrophication elsewhere, and that no eutrophication had been found.
- The Engineering and Water Supply Department report noted that the impact of nutrients in stormwater during the winter months probably exceeded that of nutrients from the treatment works. (Comparative figures by Kinhill etc indicate that the treatment works discharge could be expected to have by far the greater effect.)
- An excess of epiphytic growth is more likely to shorten plant lifespans than to cause regression (Engineering and Water Supply Department 1975). Even if such growth had resulted from a supply of nutrients, it might not account for either the initial regression or the present lack of colonisation of affected areas.
- High levels of epiphytic growth can also occur when seagrass growth is retarded for some other reason, such as possibly through discharge of waste herbicides or other toxins.

The points from EWS 75/14 discussed above have been re-evaluated by the Engineering and Water Supply Department on the basis of further studies. That department now considers that excess epiphyte growth could be a possible cause of seagrass degradation in the immediate vicinity of the Glenelg Sewage Treatment Works effluent. However, it considers that the majority of seagrass degradation appears to be due to other causes such as sedimentation and scouring (EWS comment on a draft of this report 1984).

Conclusions

The foregoing clearly demonstrates the inadequacy of present knowledge about the seagrasses and that it is not possible to draw firm conclusions on the reasons for the regression. Similarly, impacts that further regression may have on the coastline or on protection strategies cannot be determined.

It would seem that epiphytic growth associated with nutrient discharge (mainly from the Glenelg Sewage Treatment Works) was probably an initial factor and continues to influence the situation. However, the recent large losses are more likely due to movement of sand that has been progressively released as the seagrass has been lost.

Further seaward regression may be limited as the seagrass edge moves into deeper water, where wave bottom effects are less. There is some evidence that the regression rate has decreased since 1972–75, possibly for this reason.

A forthcoming report by the Engineering and Water Supply Department on recent studies of the area off Glenelg may throw more light on the matter. However, an adequate understanding is unlikely without considerable detailed research into all the relevant factors as discussed in this section. A comprehensive research project on this has recently been approved for Australian Government funding, by a combined Department of Fisheries, University of Adelaide and environmental consultant group, and will be starting in early 1984.

Depending on the results of future research, it may be necessary to reconsider aspects of the protection strategies considered in this report. At present the possible effects of seagrass regression must be borne in mind, but cannot be directly taken into account.

CHAPTER 3: THE EROSION PROBLEM

The physical processes described in Chapter 2 explain why Adelaide has a coast erosion problem, and why the coast will respond in different ways to various coast protection measures. The size of the problem and its future implications will now be considered.

A general assumption is made that erosion will be allowed to continue, and that seawalls will not be replaced or repaired. Although not realistic, this simplifies putting the erosion risk in dollar terms, and allows existing protection works to be taken into account. Some mention is made, where necessary, of the effect of various strategies, particularly continuing the present one. However, discussion on these is mainly deferred to Chapter 5.

From Chapter 2, it is clear that the erosion problem should be considered in the following context:

- Sand is being lost from the southern beaches at an average rate of approximately 35,000 m^3/year. (This allows for both the wave-driven littoral transport and a 2 mm/year relative sea level rise.)
- The sand budget for the central and northern parts of the study area is relatively stable, but these are subject to storm erosion of seafront dunes, end effects of seawalls, and some natural variation in shoreline position.
- Only a portion of the present seawalls are capable of withstanding anticipated beach drop and the larger waves that the deeper water would allow to reach the seawalls (refer to Section 3.3).

The major problem is the threat to residential development and esplanade roads. This is dealt with first, and the Alternatives Study estimates of property value at risk is set out and discussed. A lesser, but not insignificant, aspect is the higher risk applying to some beach facilities and clubhouses, which are closer to the sea, and which invariably are the first to be damaged. These and other special problems are dealt with under separate headings, though the value of such property has not been separately estimated.

3.1 RESIDENTIAL DEVELOPMENT AND ESPLANADE ROADS

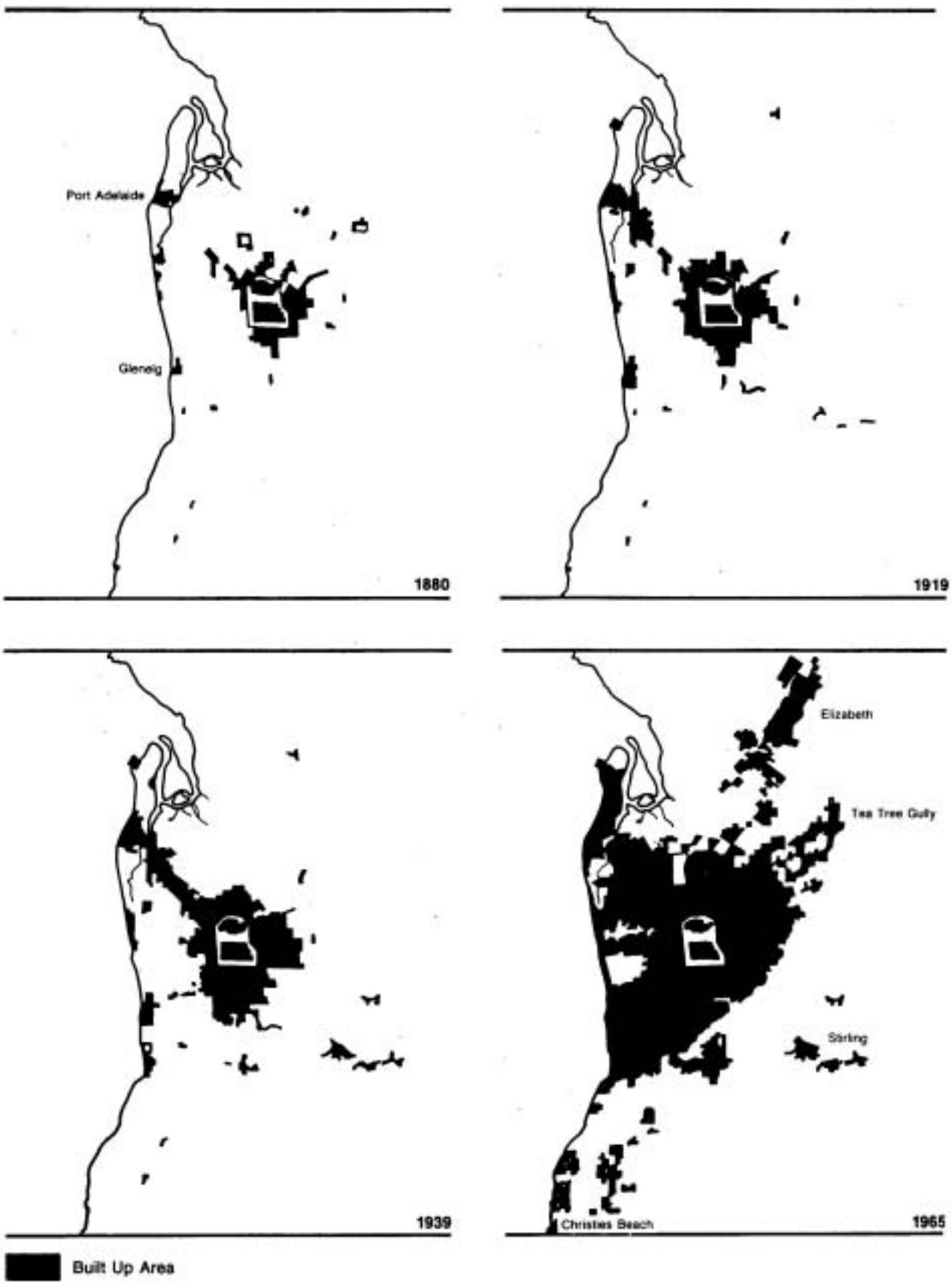
3.1.1 Development History

Progressive development of Adelaide and its coastline is shown in Figure 46. Development at Glenelg and Somerton occurred soon after initial settlement in 1836, and by 1870 to 1900 substantial coastal development had occurred at these two places and in pockets between them.

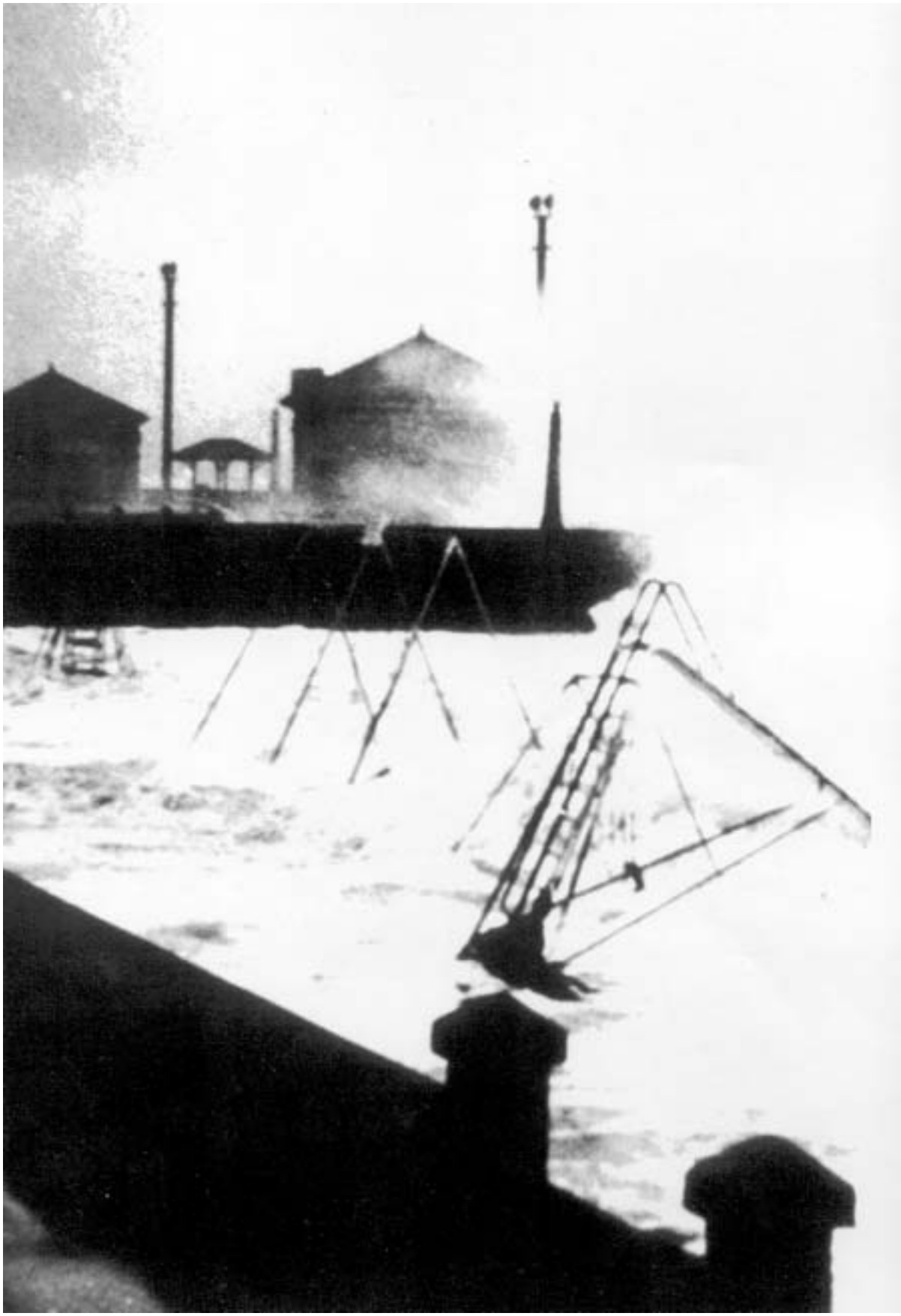
As with development to follow, coastal sites were favoured because of their elevation above the areas of swamp and low bushland, and because a seaside location was presumably as much sought after then as now. With few exceptions, an esplanade road was provided on the dunes immediately behind the beach, and the housing allotments laid out in rectangular grids behind. At some places the esplanade roads were never built, and have since been developed as public car parks (eg at Gladstone Road, Brighton), for public reserves (sometimes with toilet or other facilities on them), and in many cases (eg between Marlborough Street and Grange Road, Henley Beach) have been at least partly lost through erosion.

Land subdivision escalated in the early 1900s, and most of the problem areas (in the southern part of the study area) had been divided by 1930. Although there was substantial development at Port Adelaide and Semaphore, subdivision of the central and northern parts of the study area lagged that to the south. Most of the subdivision at North Glenelg, West Beach and between Grange and Semaphore occurred between 1930 and 1960, by which time virtually all the coast of the study area, except that at West Lakes, had been subdivided and committed to the building of houses very close to the beach.

FIGURE 46 ADELAIDE DEVELOPMENT HISTORY (WILLIAMS 1974)

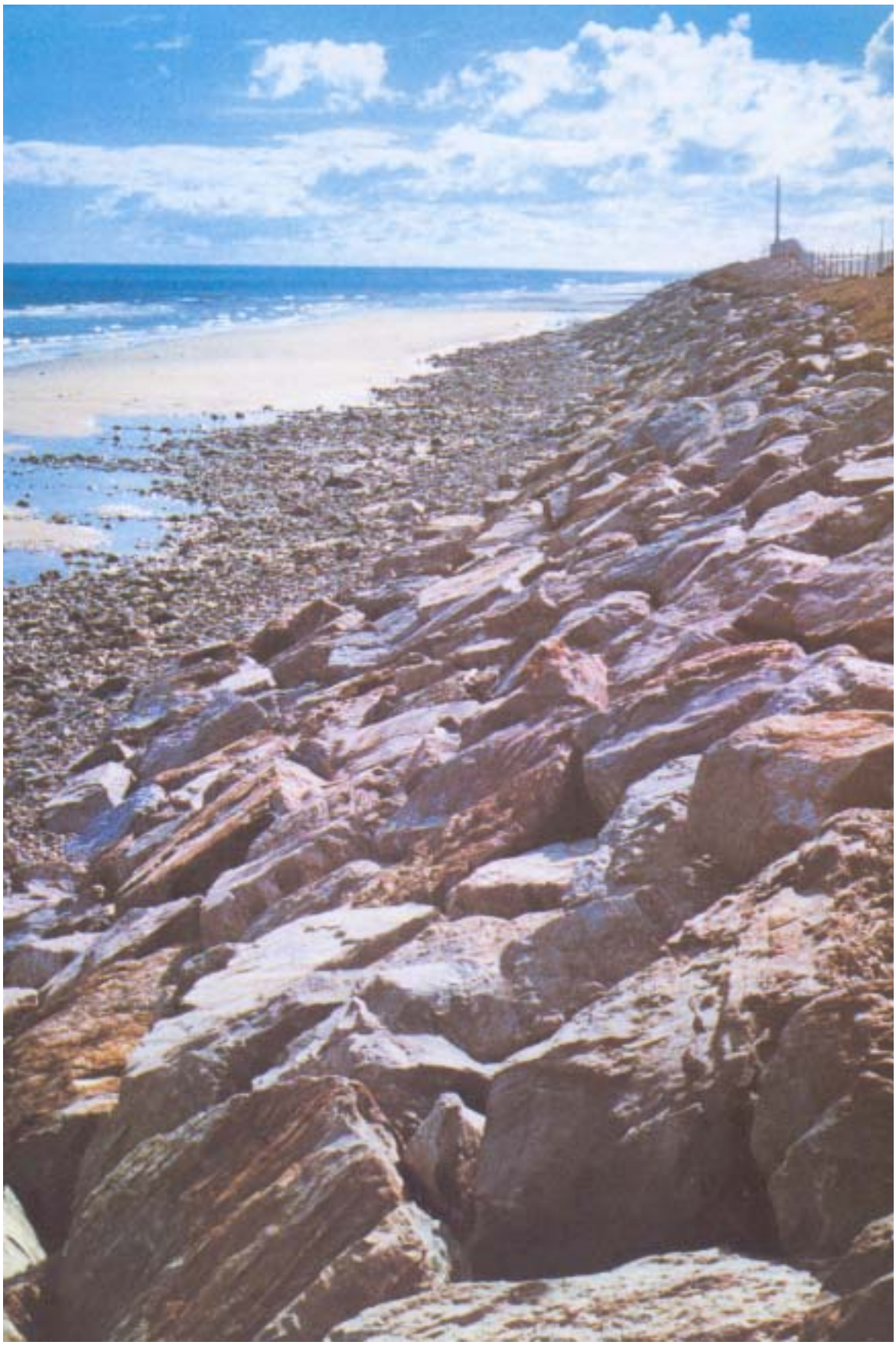


PHOTOGRAPH GLENELG (STORM), MAY 1953



PHOTOGRAPH HENLEY STORM DAMAGE, 1953

PHOTOGRAPH SOMERTON, JUNE 1980



PHOTOGRAPHS HENLEY BEACH, STORM DAMAGE, JULY 1981; HENLEY BEACH, SEAWALL AND NEW DUNE, 1982

It should be noted that the coastal processes were not understood at the time and that this development was carried out in the belief that it was the right and progressive thing to do. Unfortunately, this attitude still prevails to a certain extent.

The seafront part of the West Lakes development has proceeded since the West Lakes Indenture Act of 1972. Some of the earlier development is rather close to the sea, and may be subject to future risk, but housing that is more recent has been set further back following Coastal Management Branch comment at the development approval stage. A part of the Estcourt House dunes, forming a portion of the scheduled West Lakes development, represents the last substantial parcel of vacant privately owned land in the study area. This land adjoins land purchased for conservation by the State Government in 1974 (with the assistance of a National Estate grant).

The present situation is that virtually all of the subdivided seafront land has been built on to the extent that there is insufficient vacant land for public needs. The Board has assisted councils in purchasing suitable vacant allotments for parking and reserves and even allotments due for redevelopment have been offered and considered.

In recent years emphasis has been on redevelopment, mainly to replace older houses with medium density apartment blocks, and a few high rise proposals have been put forward. All proposals to date (mainly in the Glenelg area) have been forwarded by the councils for Coast Protection Board comment. The proposals have been in established areas behind seawalls and, in most cases, behind the esplanade road. The Board has not opposed this development, though has, where it considered appropriate, stipulated that foundations be capable of withstanding failure of the seawall and loss of some of the original dune sand. In some cases the Board has also commented on planning and aesthetic aspects, and has endeavoured to have excess sand (from excavations) placed on the beach.

The trend towards development of medium and high rise residential buildings to replace older houses is important because of the added investment behind seawalls, which may need replacing. However, such development is unlikely to affect the need for or timing of seawall reconstruction, which can be justified on the present development (see Section 3.2). Higher density development may actually be desirable, because it provides more council revenue for coast protection.

Services (water supply, gas, electricity and telephone) are usually located in the esplanade road, and add to the investment at risk, and to the disruption when the road is damaged during a storm. In at least two places (Kingston Park and near Reedie Street, Henley) sewer mains are located very close to the sea in esplanade road reserves that have not been developed as roads. Part of the sewer through Kingston Park has been relocated following coastal erosion in 1981, and that at Henley has been protected by the recently constructed seawall (1982).

Other coast protection measures to protect threatened property are described in the next chapter. In one instance, at Kingston Park, a single house was threatened throughout the 1970s and was left at extreme risk after the 1981 storms. The property was subsequently purchased by the Government and the house demolished. Another house and part of a public reserve are still under threat, but the beach replenishment program provides some protection, and no action is proposed other than possible future purchase of the remaining property.

As in the previous instance, purchase would be mainly to consolidate the public reserve rather than as an alternative to protection. The Government policy is that Government funds be used only for the protection of public property, or the community at large, and that protection of private property is otherwise the responsibility of the property owner.

3.1.2 Value of Property at Risk

An estimate of the value of property at risk is needed for any objective assessment of the erosion problem, and to put the costs of coast protection into context. However, a realistic value of future property loss is very difficult, if not impossible, to obtain because of the necessary assumptions. For example, estimates or assumptions need to be made on the following very uncertain aspects:

- erosion rates, and the way these will affect different parts of the coast;

- the stage at which property should be assumed to be 'lost', the extent to which this may be affected by earlier loss of services, and whether or not these could be provided by alternative routes to extend the useable life of the property; and
- the adequacy of existing seawalls, the degree of protection provided after they have failed, and whether or not they are repaired or replaced.

A full investigation of all the possible scenarios, both realistic and hypothetical, would be extremely complex and of doubtful value. What was needed was a reasonable order-of-cost figure to give some scale to the problem, and to take into account with the even less quantifiable social costs.

Kinhill Stearns and Riedel & Byrne did such an evaluation as part of the Alternatives Study. They assumed a uniform rate of coastal recession, based on loss of sand due to alongshore transport and a small relative rise in sea level, and also that there would be no future coast protection measures of any kind – not even repair of damaged seawalls. They assessed the adequacy of the existing seawalls (see Section 3.3), and took this into account in deriving the erosion rates. Road and property losses were calculated for 5-year intervals, with the value of lost services being included at the time when property was eroded and residents had to leave.

Although fraught with hypothetical assumptions, this approach provided useful figures that are appropriate to the scope and purpose of this study.

The consultants used 1980 Valuation Department values with a 20% allowance for inflation to 1982. Values for roads and services were obtained from the appropriate departments and instrumentalities. Using these, they estimated a total property loss of $28 million over 50 years, with a present value (using a 5% discount rate) of $10.3 million. Most of this loss would occur after the first 20 years.

Erosion would commence at Seacliff and South Brighton where the 1950s dumped rock would offer little protection once the beach replenishment ceased. This loss was estimated at $8.5 million, mostly in the first 30 years. The erosion would progressively move northwards through Brighton and South Glenelg, but losses would be less here and would be deferred because of the better seawalls.

The consultants estimated considerable losses ($8.5 million) at Henley Beach, between the Torrens Outlet and Grange, occurring over the next 30 years. Serious loss of property would occur in the West Lakes area ($6.5 million in 40 to 45 years) and at Tennyson, some buildings at these two places being within 15 to 20 m of the beach.

Because of the aforementioned limitations of this exercise, only the broad findings are quoted here. Further information is available in the Alternatives Report.

3.1.3 Controls over Development, Taking of Sand, and Access

Although not directly related to the main subject of this chapter, it is useful to follow the discussion of development with a brief review of the controls available to prevent building on eroding parts of the coast. Controls over access into sensitive dunal areas, and over removal of sand from beaches, are also addressed in this section.

The main controls are through the state's planning legislation (Planning Act 1982) and through the Coast Protection Act 1972. The Harbours Act, the Local Government Act and the Mining Act also contain important provisions. Other State and Commonwealth legislation also has a bearing on management of the coast, but has less relevance to the erosion problem and the strategies addressed in this report. More information on the legal and administrative aspects of coastal management in South Australia is provided in a Coastal Management Branch technical report 'Coastal Zone Management in South Australia – Position Paper, January 1983' (Wynne 1983). Only the most relevant aspects are reviewed here.

The Coast Protection Act

The Act allows for regulations to be made to control 'prescribed works' within coast protection districts. In practice, the need for such regulations has not been sufficiently urgent to warrant the introduction of controls that would overlap those of the planning system. The regulatory powers within the Coast Protection Act could nevertheless be invoked if other controls are found inadequate in a particular situation.

A less direct but more important development control aspect of the Coast Protection Act is the requirement that management plans be produced for Coast Protection Districts. The Management Plan for the Metropolitan Coast Protection District contains general policies, many of which are intended to prevent development that might lead to future problems. The planning system allows for coastal management plans to be adopted as Supplementary Development Plans, but the Board has resolved not to use this provision, because of basic differences between the intentions and purposes of SDPs and Management Plans. The management plans can also provide a certain degree of legal authority for the Board to undertake certain actions such as the beach replenishment program.

The Act enables public land (excluding roads) to be declared to be within 'restricted areas'. The areas must be fenced, marked and signposted, and the restriction approved by the Minister and gazetted. Persons found guilty of contravening a restricted area prohibition can be fined up to $50.

Restricted areas have been declared over sand dunes within the study area and, having regard to the importance of these dunes for coast protection, more restrictions may be needed.

Consideration has also been given to using the restricted area provision to legally close beaches when needed for public safety during sand carting or other coastal works. Unfortunately much of the beach forms part of the esplanade road reserve and cannot be included in restricted areas at present. Steps are under way, in association with the seaside councils, to have the seaward boundaries of the roads redefined to exclude the beach and dunes, and to have these latter areas declared as separate reserves under the care of councils.

The Planning Act

It is only practical to provide a short resume of the most relevant parts of the Planning Act. More information may be obtained from the guide to the Act and the procedures manuals prepared by the Department of Environment and Planning in 1982, or by reference to the Act and Regulations.

Before considering the details, it is worth noting that the Act seeks to distribute responsibility for the control of development between State and Local Government, on the basis of what is considered to be of local or state concern. The result is that most coastal planning decisions are made by local councils and, with exceptions that will be noted, only matters of state importance are decided at the state level – by the SA Planning Commission. The Commission is a statutory body established by the Act.

The most important parts of the Act and the regulations are:

- the definition of development;
- the Development Plan and procedures for amending it;
- provision for certain works in certain areas of the state (eg the coast) to be referred to the Commission;
- notification required of Government and semi-Government works, such as those that might be undertaken by the Board; and
- the provision for Environmental Impact Statements.

Development is defined in the Act to include land division, building, road construction, change of land use, and prescribed mining. However, it should be noted that council construction and maintenance of roads, drains and pipes is specifically excluded from development in the first schedule of the regulations. This schedule also excludes retaining walls lower than one metre and mining where this is conducted under the Mining Act. The inclusion of other mining may enable the

Act to be used to prevent the removal of sand or other material from beaches, though separate control under the Coast Protection Act may be preferred. Dredging, which can have major coastal impacts, is not included in 'development'.

The Development Plan, which sets out objectives, proposals and principles of development control, forms the basis of planning decisions. In defining 'permitted', 'prohibited' and 'consent' uses, the plan has a bearing on which coastal proposals are to be referred to the Planning Commission and hence to the Board and the Branch for comment. The Development Plan can be varied by Supplementary Development Plans and is kept current through these. Draft SDPs, after acceptance by the Minister, must be advertised for public comment. This is followed by a specified procedure for considering the comment and making amendments before the SDP is authorised. Policies on coastal issues would generally be prepared after discussion with officers of the Branch and may be referred to the Board. They would draw on recommendations of the various District Study Reports and on Management Plans where these are available.

An SDP is proposed for Torrens Island where timely resolution of future land uses is essential. The proposed SDP will be of special coastal management and coast protection interest because the island contains the most important known source of beach replenishment sand, and because of the high conservation value of the samphire and mangrove environments elsewhere on the island. Other important uses that also need to be taken into account are possible extension of electricity generation and port and industrial uses.

The Act enables regulations to define which developments are to be decided by councils and which by the Planning Commission, and which are to be referred to the Commission for consultation. These **referral and consultation procedures** are very important to coastal management. Only certain 'consent' applications need to be referred to the Commission for consultation, and this applies only to some types of development in some parts of the state. These are defined in the fifth schedule of the regulations. The most relevant here are the inclusion of land division as a type of development to be referred, and the inclusion of the coast as an area within which 'consent' applications must be referred. For this purpose the coast is defined as any of various types of coastal zone delineated in development plans, or as land within 100 m of high water mark (urban areas) or 500 m of mean high water mark (rural areas).

This referral procedure is important to the Board as it enables the Board and the Branch to provide comment and, where appropriate, to recommend either against proposals or for special conditions. All coastal development proposals forwarded to the Planning Commission are sent on to the Branch for comment and may also be considered by the Board. The chairman of the Commission is also, by virtue of the Coast Protection Act, chairman of the Board, and this provides an important link.

Councils must have regard to the advice provided by the Board or Branch through the Commission, but are not obliged to accept it. For the study area the developments submitted are usually only those involving a change from a single unit residential use to a different use or a more intensive residential use. Replacements of buildings, additions, and proposals that conform to the established zoning do not have to be referred to the Commission or the Board. In practice, councils may seek the Branch's advice for developments that do not require consultation with the Commission.

Private development proposals referred by councils would usually be either behind the esplanade road or behind existing seawalls. Since the latter are likely to continue to be maintained, the coastal hazard to such developments is not likely to be high. However, special building foundations might be required to give added protection for the higher investment, as in the case of high rise buildings. Building site levels and floor levels may also be critical in some of the lower, northern parts of the study area.

The seventh schedule of the regulations lists development by councils and prescribed mining among 'consent' applications to be determined by the Commission. This also includes development by clubs or others on land under council care control and management. It should be noted that council road and drainage works are not 'development', as discussed, and do not

have to be approved by the Commission. Virtually all development in the high-risk area seaward of the esplanade roads on the Adelaide coast would be on land under council control and would be decided by the Commission.

Development by the Crown is dealt with in regulations 58 and 59, which include the Coast Protection Board among Crown agencies to which Section 7 of the Act applies. Section 7 requires the Crown instrumentality or agency to notify the Commission and the relevant council of its proposed development. The council may report to the Commission on the proposal, and the Commission must report to the Minister, who is required to have copies of the report laid before both Houses of Parliament. Regulation 59 sets out types of work for which notice is not required. These include various essential and emergency types of work, but do not include either planned or emergency coast protection works.

Coastal defence structures are usually built by councils as council projects (albeit with substantial financial and technical assistance from the Board) and consequently need the Commission's approval. If the Board were to provide these as its own projects, as is allowed for in the Coast Protection Act, it would need to notify the relevant councils and the Commission. The metropolitan beach replenishment program has been considered neither development nor mining and, although councils are always notified, does not require planning approval.

The Planning Act enables the Minister to call for **Environmental Impact Statements** for developments or projects that are, in his opinion, of major social, economic or environmental importance. It also provides for public comment on these statements and for them to be given due weight by the planning authority. Depending on the choice of future coast protection strategy, the drawing up of an environmental impact statement may be considered appropriate. Either this or the informal procedure of providing a Public Environment Report may be appropriate to mining of the Torrens Island or other inland sand deposits for beach replenishment (See Section 5.4).

The Harbours, Local Government and Mining Acts

The Harbours Act, among other things, provides for the placing of the foreshore under the care, control and management of local councils, Ministers or the Coast Protection Board. The Act automatically places foreshores under the control of councils where they are not under the control of some other authority. In the study area, all foreshores are under the control of councils except where specifically removed by Act or Parliament, such as for the West Beach Trust and at North Haven.

The Local Government Act provides the rules as to how Local Government may administer the coast, providing the scope for by-laws and also special conditions for the removal of beach material or for the licensing of this removal by others. Although councils may remove beach material for their own purposes, most are now well aware of the coast protection value of beach sand and have acceded to the Board's request that sand should not be removed. Councils may license the removal of material, but are required to use the revenue obtained from this for foreshore improvements. This is now of academic interest only as metropolitan councils no longer permit the removal of sand.

The Mining Act has little relevance to the study area except through the penalties that it provides for illegal mining, which includes the taking of sand from beaches, whether for private use or for sale. The Board has obtained legal advice that its taking of beach sand for replenishment within the same Coast Protection District does not constitute 'mining'. It is awaiting a further opinion about the use of sand from the deposit on Torrens Island, though it appears likely that the Board would need to obtain a mining tenement, either in its own right or through ETSA, the owner of the land. A strip of the coast 800 m inland from high water mark is reserved from the provisions of the Mining Act, and special procedures, including Cabinet approval, are needed for this reservation to be lifted before a mining lease can be obtained.

Removal of Beach Sand for Replenishment

There has been lack of agreement with some councils about the removal of sand for beach replenishment, and there may be future difficulties in this area. Legal opinion, based on the legislation discussed, has been that councils have no authority to prevent the Board from removing sand, and that the Board has an implied authority to carry out its replenishment program. An amendment to the Coast Protection Act would be required to make that authority specific.

Preventing the Removal of Sand

The authority for councils to remove sand and to license others to do so has already been discussed. Although councils have generally acceded to the Board's requests not to allow sand to be removed, this agreement has not always been unanimous. As a result the question has been raised as to whether it might be necessary for the Board rather than councils to control the beach sands, and whether or not the Board should be empowered to prevent councils or others from taking sand.

It may be possible to achieve such control under the Planning Act, though more direct control by regulation under the Coast Protection Act could be preferable. This has been under consideration for several years, but has not proceeded, as it has been the Board's view that it is imperative to obtain voluntary agreement from councils on such matters.

Adequacy of Controls

This study has not attempted a rigorous assessment of the adequacy of controls available for avoiding inappropriate development or for facilitating coast protection measures. However, from the foregoing brief description and discussion of the legislation, and from the Branch's experience of working with it, it can be concluded that existing legislative provisions are adequate to prevent most future problems without unduly restricting the timely provision of coast protection measures. The Planning Act is still considered to have a few minor deficiencies, however, as listed below. The Board has already raised these with the Planning Act Review Committee, which has considered some of them. Others are still under consideration. Those having a bearing on the metropolitan coastline are:

- The non-inclusion in Regulation 59 of emergency Government coast protection works from the lists of those Government works exempted from the requirement that councils and the Planning Commission be notified. Other urgent Government works such as for water supply or for road and bridge repairs are exempted. Difficulties would only arise when it became necessary (because of possible lack of council agreement or lack of council funds) for the Board to carry out emergency work. In such cases, a 2 to 3 month delay may be untenable. The Review Committee has recommended an appropriate amendment.
- The exclusion of council road and drainage works from the definition of 'development', and consequently from the provisions of the Act. Esplanade roads are commonly at more risk than other coastal property, and stormwater outlets especially need to be taken into account when considering a coast protection strategy.
- The exclusion of retaining walls less than one metre in height. Even such low structures, if built as seawalls, can interact with coastal processes, causing loss of beaches and erosion of adjacent property.
- The omission of dredging and earthworks from the definition of 'development'. Dredging, in particular, can damage a coastal environment and, if done close to shore, can influence coastal processes, possibly causing erosion.
- A lack of clarity as to whether or not the removal of beach sand constitutes 'prescribed mining' as defined in the Act and controlled by it.

The desirability of amending the Coast Protection Act to give the Board specific authority to remove beach sand and carry out beach replenishment has already been discussed, as has the possibility of a regulation under the Act to prevent the removal of beach sand by others.

Whatever the control measures, administrative arrangements and cooperation between the relevant bodies are likely to have an important bearing. Good communication between the councils and the Department and particularly between the Branch and the Metropolitan Branch of the Development Management Division of the Department is essential.

The coastal management input may be only one of several in the decision-making process and there is no guarantee that it will always prevail, even though this may seem imperative in some situations. Continued effort will therefore be needed to ensure that development plan policies give adequate regard to coastal issues, and that coastal erosion aspects are given due weight in the consideration of individual applications.

3.2 OTHER COASTAL STRUCTURES

Structures associated with beach recreation are usually placed closer to the sea than other buildings, and are invariably the first to be damaged. Boat ramps, jetties, seawater swimming pools, and clubhouses for sailing and surf rescue clubs are often at high risk. The study area contains all of these, though some are behind existing seawalls and others are in the northern part of the study area, where the risk of damage is less. The potential losses are worth identifying, though the monetary values are in most cases not high.

3.2.1 Jetties

The Semaphore jetty and the original timber Glenelg one were built between 1856 and 1860, and the others (Brighton, Henley, Grange and Largs Bay) were built between 1880 and 1883. The Glenelg jetty was reconstructed in concrete in 1969. All the others were of the original timber construction.

Some of the jetties were formerly equipped with bathing sheds, kiosks, side-shows and other amusement buildings, but storm damage to these, especially at Glenelg, has discouraged this use, which has also gone out of fashion. The six jetties are now maintained to minimum structural standards for fishing and promenading.

Decking damage often occurs during minor storms (most often at Brighton, this jetty being slightly lower than the others), and structural damage can be expected at any of the jetties during major events. The concrete Glenelg jetty is a possible exception. Storm damage repair to a typical timber jetty could be expected to be in the range $40,000 to $100,000, assuming structural damage. In the event of total damage, any replacements would most likely be to a different design (either in concrete or steel) to minimise costs and to avoid the high maintenance of the timber structures. Replacement costs might be in the range $0.2 to $0.5 million a jetty, depending on the length.

The erosion risk to the jetties will not be influenced by the choice of protection strategy, except perhaps at Brighton and to a lesser extent at Glenelg, where a major beach replenishment would cause some decrease in wave height for a portion of the jetty length.

3.2.2 Public Boat Ramps

The two major ramps in the study area are at Glenelg and North Haven. These are both protected by substantial structures, and are not vulnerable to coastal erosion. The other ramps are for boat access onto beaches (for subsequent launching from the beaches) and are consequently dependent on the beach levels. The ramps at Seacliff, Edward Street (South Brighton) and Grange Road depend on beach replenishment, and would soon be lost if this were stopped. When damaged, they would need to be replaced with more substantial structures to withstand the greater waves caused by lowered beaches. The only other significant ramps are those at the Holdfast Bay Sea Rescue Squadron and Point Malcolm. The former is already damaged and, having proved unsatisfactory and little used, has now been virtually abandoned. Access for beach launching is gained via a nearby rubble ramp. The ramp at Point Malcolm is at a less vulnerable part of the coast, though it may need to be modified if the beach replenishment strategy

continues using sand from this source. Recent provision of a public ramp at North Haven reduces the importance of the Point Malcolm ramp for power boating, though it may have a continued important use for sailing dinghies.

If the beach replenishment program is not continued, loss of the beaches that are used for launching, rigging and parking trailers would be of more consequence to boating than damage to the ramps would be. However, continued or increased beach replenishment may also affect use of the Seacliff and Edward Street ramps, which may need to be extended across the reservoir of replenishment sand.

3.2.3 Surf Rescue and Sailing Clubs

Several of the surf life saving clubs (SLSCs) and sailing or yachting clubs (YCs) have clubhouses in vulnerable or potentially vulnerable positions. This section provides a brief review of their situation.

The main activities of these clubs benefit from being sited as close as possible to the beach, but unfortunately these special activities are invariably combined in one substantial building with the club's other facilities, such as bars, clubrooms and toilets, which would be better located separately further back from the sea. Future policy should be to split the two functions where practical, with the essential elements being provided in smaller, expendable or removable structures, and the social functions being further back, preferably behind the coastal road. However, such a policy is likely to be constrained in the study area by the lack of available land and its high cost.

The buildings of the Brighton and Seacliff YC and the Seacliff SLSC are at the greatest risk, because these depend on continued beach replenishment, and could be damaged within a few years of this being stopped. This applies especially to a recent seaward extension of the yacht club.

The Henley SLSC is probably at the next highest risk, especially its large concrete boat ramp, which is presently vulnerable. The recent building extensions are on piles to minimise the risk to the building itself, but some damage would be inevitable if the old concrete seawall were to fail. This wall has been close to being undermined several times, though could suffice for many years if beach sand levels are maintained.

Four clubs have buildings that are well forward, and which would be subject to erosion were it not for the rock protection placed in front of them. The clubs are Somerton SLSC, Holdfast Bay Sea Rescue Squadron, Holdfast Bay YC, and the West Beach SLSC. These could be subject to loss in the long term if the protection failed and was not replaced. They are at slightly more risk than housing on the landward side of protected esplanade roads.

Several clubs are no further seaward than other development, and would be no more vulnerable than this development except that they might lose their relatively inexpensive beach access ramps. Such clubs are Somerton YC, Glenelg YC, Glenelg SLSC, Grange YC, and the Grange SLSC. The Grange ramp, which was damaged in both 1980 and 1981, is shared by the yacht club and the general public, and is also used for beach replenishment.

The three clubs in the northern part of the study area (the Semaphore SLSC, the Largs Bay YC, and the Taperoo SLSC) are safe by virtue of being on an accreting part of the coast. The former Taperoo clubhouse was stranded by the build-up of sand adjacent to the North Haven breakwater, and has recently been relocated to serve the new small beach between North Haven and Outer Harbour. Although this beach is more stable than most in the study area, it is subject to seasonal and storm changes, and the new structure could be at slight risk.

3.2.4 Marineland Facilities at West Beach

The main threat is to the seawater supply to Marineland. This is comprised of a 250 mm intake pipe, a 2 m diameter concrete caisson containing the pump, and a recently built concrete water tank on the seaward side of the main Marineland complex.

The seawater supply was built in 1976 to replace the former supply, which was inadequate, and which had been damaged as a result of progressive erosion of the dunes. The caisson was located a small distance back from the dune face – a compromise having been necessary between allowing for future erosion and avoiding disrupting the dunes with an excessive length of deep and very difficult excavation. Its present location on the beach is a result of the continued erosion.

The immediate risk is less than it might seem, because the bottom of the caisson is still nearly 5 m below the beach, and the pipe to sea is also still well covered. Following advice from the Trust's engineering consultants, the caisson was modified to make it self-standing (a heavy concrete valve chamber at the top was replaced with a lighter steel one), and it has been assessed as being safe in its present position.

If erosion continues, as it will if the beach replenishment is curtailed (or as it may to a lesser extent despite present measures), the pipework and electrical connections from the caisson to Marineland will need to be provided with support. With continued erosion the caisson itself would become exposed to larger seas and could not be expected to survive. The new seawater storage tank could also be undermined, though it is unlikely that the other Marineland facilities would be affected within the 50-year period considered here.

3.2.5 Henley Swimming Pool

The seawater swimming pool at Henley was built in 1933. Although it has so far survived its beach location, it has gradually deteriorated to the point where it may not be worth keeping. The Henley and Grange Council is currently considering its future.

The concrete pool is founded on the upper part of the beach, and it was originally protected by only its seaward concrete wall. Additional protection, by placing large rocks on the beach in front of the wall, was provided 15 to 20 years ago. These rocks break up some of the storm wave energy, but do not help to prevent beach drop, which will eventually cause the wall to fail. Minor cracking of the pool has already occurred, probably due to sand being lost during storms. Scour by water escaping through cracks would cause further undermining.

The pool is considered to be at considerable risk at present, and to have been lucky to survive the 1981 storms. For some reason, possibly a fortuitous effect of offshore topography on waves from the particular directions during those storms, conditions appeared to be milder here than elsewhere in the study area. Although this area benefits slightly from the minor beach replenishment at Grange Road, and could also receive an indirect benefit from a major replenishment of the southern beaches, this may not significantly reduce the risk. This could only be done by strategic placement of a groyne or offshore breakwater (which would have erosional effects to the north) or by building a proper rip-rap wall with a filter to prevent the sand being washed out.

The pool will increasingly affect the safety of the adjacent seawall and SLSC ramp to the north as sea level rises and the beach is lost. This is because the structure would have a greater interaction with the alongshore sand movement by acting as a groyne, causing sand build-up on one side (mainly the south) and erosion on the other (mainly the north). Removal of the pool would be advantageous from this point of view.

3.2.6 The Port Stanvac–Birkenhead Oil Pipeline

The 200 mm diameter welded steel pipeline is close to the coast at South Glenelg, at the Torrens Outlet (where it crosses over on the outlet structure), and at Semaphore (where it follows the seaward edge of the esplanade road).

A significant risk applies only at South Glenelg, where the pipe is within 10 to 15 m of the seawall for a distance of approximately 800 m between The Broadway and the Glenelg jetty. The seawall here is partly an old curved concrete wall (at the Broadway end) and partly an old stonework wall that was reinforced in the mid 1970s by placing rip-rap, with a filter, against its lower part.

The pipe carries a range of petroleum products including solvents, gasoline, kerosene and dieseline. Rupture could therefore result in a severe hazard as well as pollution.

If the seawall were not maintained, a major storm could expose the pipe, leaving a portion suspended and subject to wave action. However, the problem would be anticipated and should be evident before this stage was reached and before there was any serious risk of rupture; and there would be time to bring in rocks or other temporary protection.

3.2.7 Intakes, Drainage Outlets, and Sewage Treatment Discharge Pipes

With the possible exception of the Glenelg Sewage Treatment outfall pipes, these coastal structures appear to be at no more risk than other coastal development. The location of the structures is shown in Figure 47.

There are only 3 inlet structures – the 3.5 m diameter intake pipe for West Lakes (this extends under the beach to an intake 500 m to sea), and the small intakes for Marineland and the Henley swimming pool.

The larger drainage outlets are all in the southern part of the study area, most being part of the South West Districts Drainage Scheme. The larger outlets have substantial supporting structures and sheet piling cut-off walls to prevent undermining. Because of this, they are likely to survive beach drop and storm events, which would damage some of the older rock protection. However, some failure could occur within the 50-year period considered here, and replacement costs would be high. These costs were not included in the Alternatives Study estimates of property loss.

There is no threat of damage to either of the two largest outlets, these being the Patawalonga Creek and the Torrens River.

The sludge outfall from the Port Adelaide Treatment Works passes under the beach a few hundred metres south of the main replenishment source beach. The initial asbestos cement pipe, laid in 1976, suffered breakages because of wave action, and a 900 m length across the beach and the surf zone was subsequently relaid with steel pipe. There has been no damage since then, and no reports of the pipe being exposed on the beach. Although the first pipe was laid 2 m below the beach level, the replacement was laid only one metre below the sand, presumably because the beach was lower during the second pipe-laying. It could therefore be undermined by storm changes in the beach level as well as possibly by a slight beach draw-down associated with the taking of replenishment sand from the Point Malcolm/Semaphore area. It would seem prudent to periodically check the sand cover over the pipe, and to lower the pipe if there is found to be a risk of it being exposed.

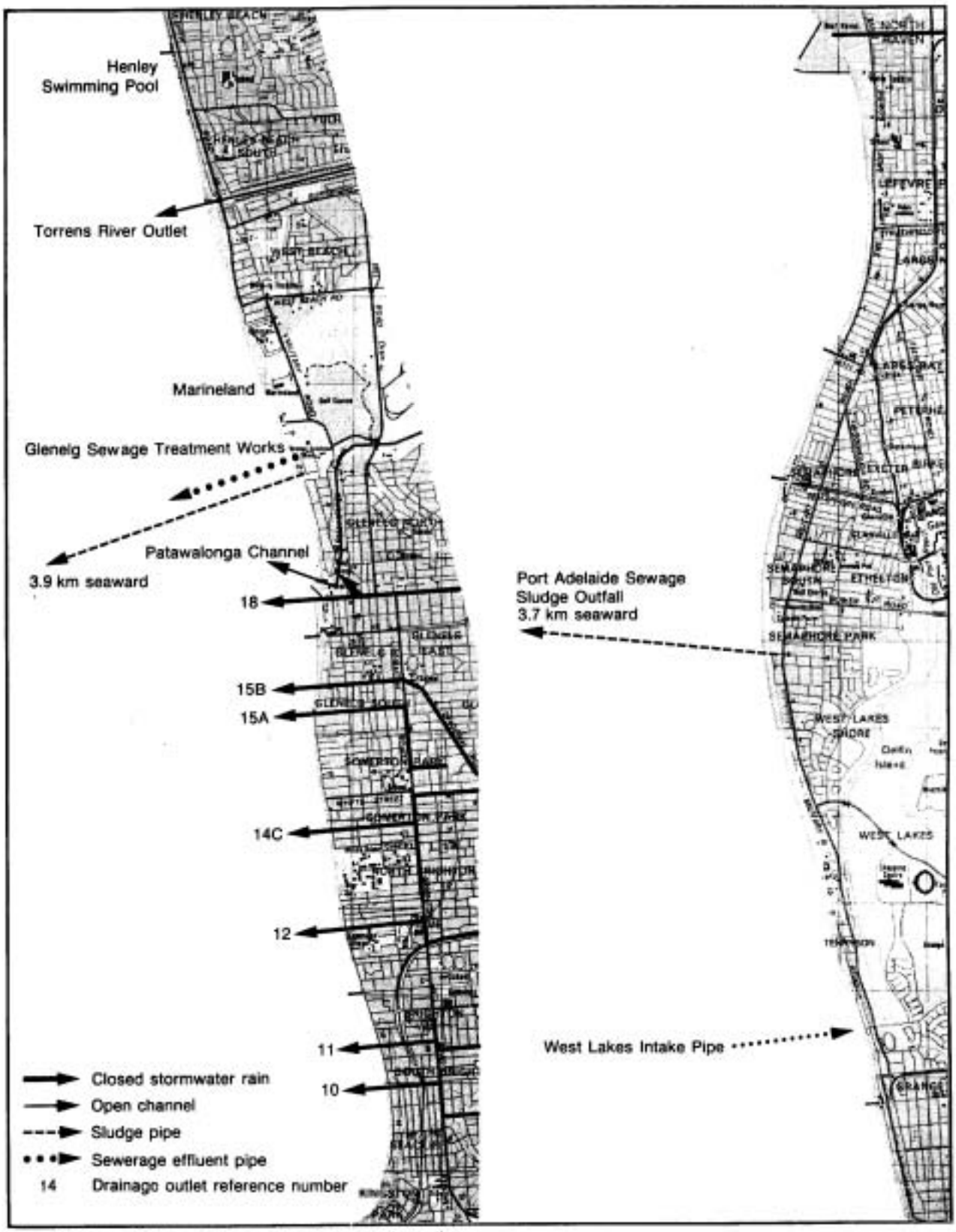
The Glenelg outfall pipes are supported on concrete piers that are founded on the clay or calcrete horizon under the sand. They are therefore unlikely to suffer directly as a result of beach drop (due to possible cessation of the replenishment program), but could suffer structural failure as a result of the larger waves associated with the beach drop. It seems unlikely that they could survive for the next 50 years in the absence of any coast protection or other measures. The value at risk may be substantial but is difficult to determine and was not included in the Alternatives Study estimates.

3.3 DAMAGE TO SEAWALLS

The seawalls themselves represent a substantial investment, which would in many cases be lost before the other property loss occurred. The Alternatives Study graded the seawalls into five main classes and estimated when various portions would be likely to need replacement, but it did not include the value of seawalls in its property loss estimates.

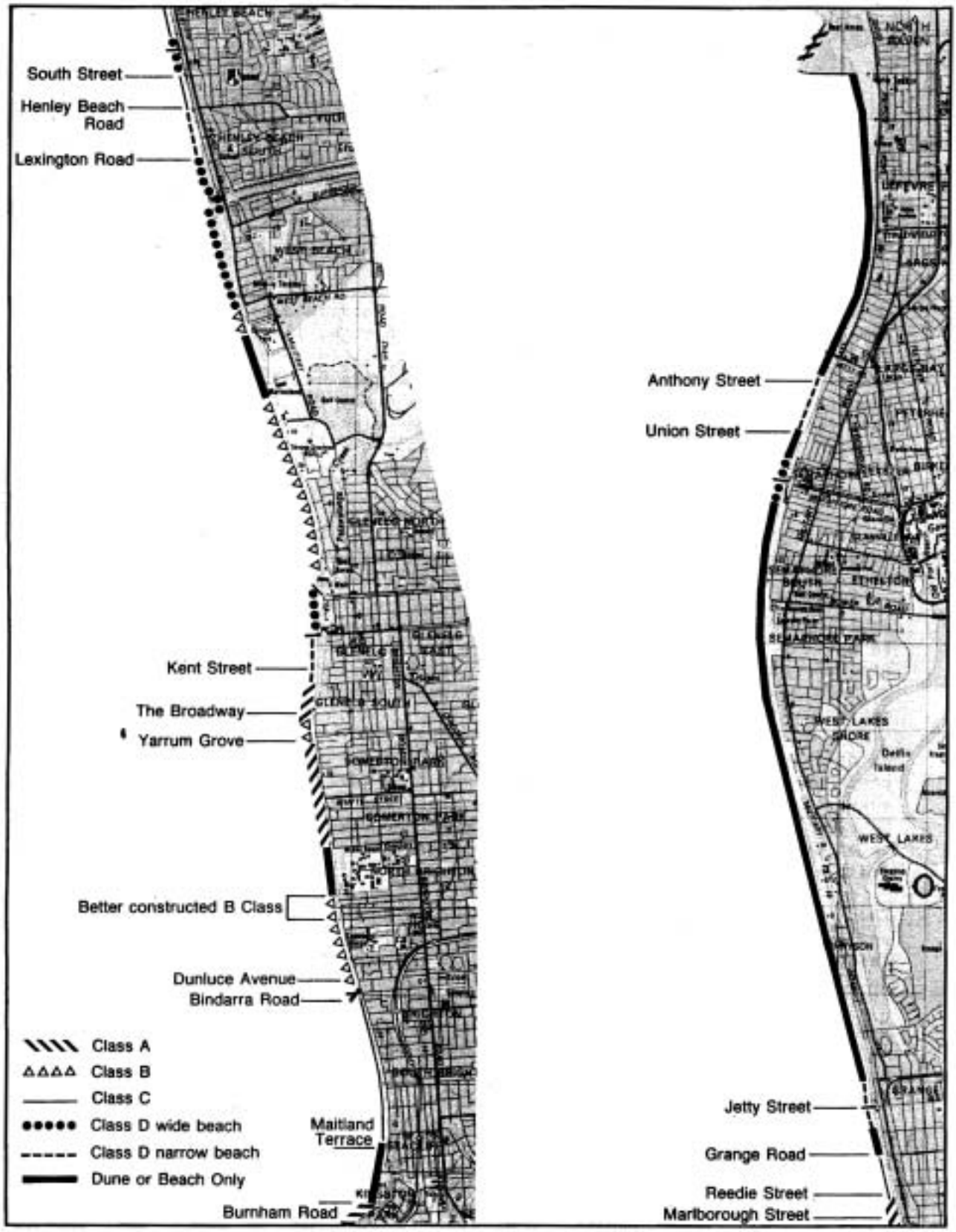
As with the previous estimates, it is difficult to make realistic assumptions; there is also great uncertainty as to when the seawalls would fail, or how complete this failure would be. Obviously, repairs would be undertaken, but it is difficult to include this in the value estimates.

FIGURE 47 INTAKES, DRAINAGE OUTLETS AND SEWAGE TREATMENT DISCHARGES



Note: Drainage Outlet Reference Numbers are from the South Western Suburbs Stormwater Drainage Scheme

FIGURE 48 CLASSES OF SEAWALL



The main consequence of no action is that beach levels would fall, causing the less substantial seawalls to be undermined and also to be damaged by the larger waves that would reach them. The Alternatives Study consultants worked closely with the Branch in assessing the seawalls, but found it very difficult to estimate how long it would take beaches to drop to the point where seawalls would fail. They concluded that the most recent design, with toe-stone (class A), would not be damaged by more than an acceptable 10% within the 50-year design period. They noted though that substantial damage could occur after that.

This may be optimistic, because it assumes an even rate of beach loss, which is unlikely. Trial excavations at Somerton, where recent beach loss has exposed the toe-stone, showed that failure was likely if the beach loss continued there. Beach loss, and the threat to the seawalls, is limited at some places by the presence of a clay layer, which drilling on Brighton beaches (see Figure 4) has shown to be too far down to prevent undermining from occurring. The horizontal 'toe' of the rip-rap seawalls is designed to settle as beach levels fall, but losses of two or more metres below the present lowest beach levels would almost certainly cause failure.

No drilling has been done at North Glenelg, but information from construction of the sewerage outlets and from work at the Patawalonga suggests that the clay horizon may be close enough to the surface there to reduce some of the risk of damage due to undermining. However, as noted in Section 4.2, the North Glenelg seawalls are at greater risk of failure due to loss of the stone filter.

The following assessment of value of future seawall losses (assuming no action) is based on the consultants' categories and their estimates of failure dates. The lengths of seawall in the various categories are shown in Figure 48.

Class A (already discussed) is unlikely to need replacement, and no loss is assumed in this category. Note that there is some uncertainty in this, and also that substantial losses would occur after the 50-year period, and possibly during this period.

Class B includes those seawalls built to a formal rip-rap design but without the toe-stone. It also includes a section of better quality dumped rock north of the Brighton jetty. 3.7 km of Class B seawall would probably fail in the next 25 years, most in the next 15. The cost of replacement to a similar standard is estimated at $3 million, based on costs of 80% of the present cost of building a similar seawall with toe-stone.

Class C consists of dumped rock along a total length of approximately 3 km, mostly at South Brighton, though there is a short length north of the Henley jetty. The consultants estimated that this would all fail within the next 10 years if no action were taken. Assuming a fairly low value of $100,000/km (the value is little more than the cost of the stone on site) gives an approximate loss of $0.3 million.

Class D mainly consists of old vertical concrete structures. Although the present day replacement cost would be high, the real value should perhaps be assumed to be between that of classes B and C – say $0.3 million for the 1.2 km likely to be lost.

Allowing for application of a discount rate to bring these future costs to present values suggests a total value in the range $2 million to $3 million. This is a substantial sum, which needs to be taken into account with the other property values discussed in Section 3.1.2.

3.4 BEACH LOSS

In the past, the rationale for protection of the coast seems to have largely overlooked the value of the beaches for recreation and as a tourist attraction. The retention of beaches for public use has been seen as a side benefit of the beach replenishment program rather than as a goal in itself. It is not surprising that this should be the case, because storm damage is more noticeable and more newsworthy than the day to day beach trips that the community takes for granted. However, it has resulted in a very important aspect not being fully appreciated.

The Alternatives Study included an assessment of beach use, and put dollar values on beaches for inclusion in the cost comparisons of the various coast protection solutions – including doing nothing. Only this 'no action' case will be discussed here. The 'beach costs and benefits' of the other options are considered in Section 5.5.1.

The consultants used data from the 1973–74 Metropolitan Adelaide Recreation survey (SA Planning Authority 1976) to derive a demand curve for Adelaide beaches, based on what people would be prepared to pay for transport to the beach, assuming 10 cents a kilometre travelled.

They obtained a figure of $3.8 million (1982 prices) for the annual value of the beaches in the study area. The value for all the Adelaide beaches, including the southern beaches to Sellicks Beach, was estimated to be $6.4 million, which compares reasonably with a similar study of the Port Phillip Bay beaches (Fisher, Pullinger and Paterson 1974), where an annual value of $16 million (1974 prices) was obtained.

It should be noted that, as with the property loss estimates, many assumptions had to be made. In addition, the basic data was very limited, and barely adequate. Nevertheless, the estimate is the best that can be made in the circumstances, and the results are not unreasonable. Although the figures may seem high at first glance, comparison with what people are prepared to pay for other forms of recreation suggests that they are of the right order. For example, the total takings from Adelaide's seven major swimming centres are approximately $0.6 million a year; and the total football gate takings in a season are approximately $3 million a year. Both of these figures would be higher if transport costs were included.

The calculation of loss of value as beaches disappear is more open to question, because of the necessary simplifying assumptions. The consultants adjusted the beach values according to distribution of people calculated from aerial photograph beach counts, and then derived dollar values per 100 m of beach length, taking into account a reduced value for beaches that were only useable at low tide. This has the effect of increasing the value of adjacent 'all tide' beaches. They concluded that an 'all tide' beach between Seacliff and Grange Road was worth $24,500/100 m/year, and a reduced beach (not available at high tide) was worth half of this, $12,250/100 m/year. Equivalent values for the beaches north of Grange Road were $6,424 and $3,212, though all beaches in this area are still available at high tide and the latter figure would apply only as beaches were lost.

A 'no protection' present day value of the future beach loss was calculated to be $23.3 million. This figure was obtained by predicting when the losses would occur, and then calculating the present day value using a 5% discount rate. It was considered that the Seacliff/Brighton beaches would be lost in 15 years, and that those between Minda and the Glenelg jetty would be eroded away within 20 years. Small beaches would remain at Minda and south of the Glenelg groyne. The North Glenelg beach would disappear quite rapidly. A beach would remain at West Beach, but the Henley Beach to Grange area would lose its beaches within about 30 years. The consultants concluded that 11 km of beach would be lost in 50 years, and that the beaches that would be lost account for 80% of the present use.

The study was unable to take account of non-linear effects that could influence the present day value. For instance, early beach losses would not be felt as much because people would be able to use other beaches. However, the loss of these (by then very crowded) beaches would have a greater impact. The effect would be to defer the losses, thus reducing the present value, though it is obviously hardly worth taking the assessment to this level of precision. As with the property loss estimate, the order of value is valid and useful, though the detailed costs remain open to question.

For the purpose of this section, it is sufficient to conclude that the taking of no action would result in progressive and quite rapid loss of a recreational resource valued at approximately $3.8 million a year, and that the larger part of this, or at least the more valuable part, could be lost within 30 years. This considers only the local recreational use, and excludes the tourism value. The beaches are undoubtedly one of Adelaide's major tourist attractions, but the effect of loss of beaches on tourism is too difficult to quantify. It would undoubtedly run into millions of dollars, with patronage of all seaside holiday accommodation and the large West Beach caravan park complex being

affected. The Department of Tourism, in commenting on the Alternatives Study, stressed the importance of the beaches for tourism. The Department of Recreation and Sport also commented, supporting the high recreational value of the beaches.

CHAPTER 4: PAST AND PRESENT PROTECTION METHODS

As mentioned in Chapter 3, coast protection works became necessary in the early days of settlement. Vertical and curved concrete walls were built, damaged by storms, and rebuilt. Later these were replaced with dumped rock, which has in turn been progressively replaced with designed rip-rap rock walling. Meanwhile there has been a progressive extension of the seawalls, as the natural supply of sand from dunes has been cut off, increasing the rate of loss of sand from the beaches and erosion of the unprotected coastline. The Board's main strategy of beach replenishment has been superimposed on this, and has gradually superseded, though not entirely eliminated, the historic trend of seawall construction.

The Branch has reviewed early protection history (Moulds 1981), and a technical report providing details of all protective works constructed since the Board's establishment in 1972 is in preparation. A report on the beach replenishment program between 1973 and 1982 has also been prepared (Penney 1982). Discussion here will be more general, and will only refer to the most relevant details from these technical reports.

The Alternatives consultancy considered the adequacy of the present seawalls, taking into account possible future beach loss. The findings are noted in Section 3.3, and discussed later in considering the alternative strategies (Chapter 5).

The progressive building of coast protection works is illustrated in Figure 49.

4.1 EARLY COAST PROTECTION WORKS

There is little detailed account of the first seventy years of European development of the coast. Most of the protective structures were ad hoc and constructed by various authorities or private individuals. The detail of construction or design is not available for many of these. In the period 1900–50, construction of protective works was carried out in the main by local councils or the South Australian Harbours Board, with no overall coordination. The records of design and costing are also difficult to ascertain. After 1950 more detail is available, particularly from the late 1960s onwards when the establishment of coordinating bodies such as the Seaside Councils Committee, and later the Coast Protection Board, resulted in more systematic protective measures being undertaken, with a uniformity of design and construction.

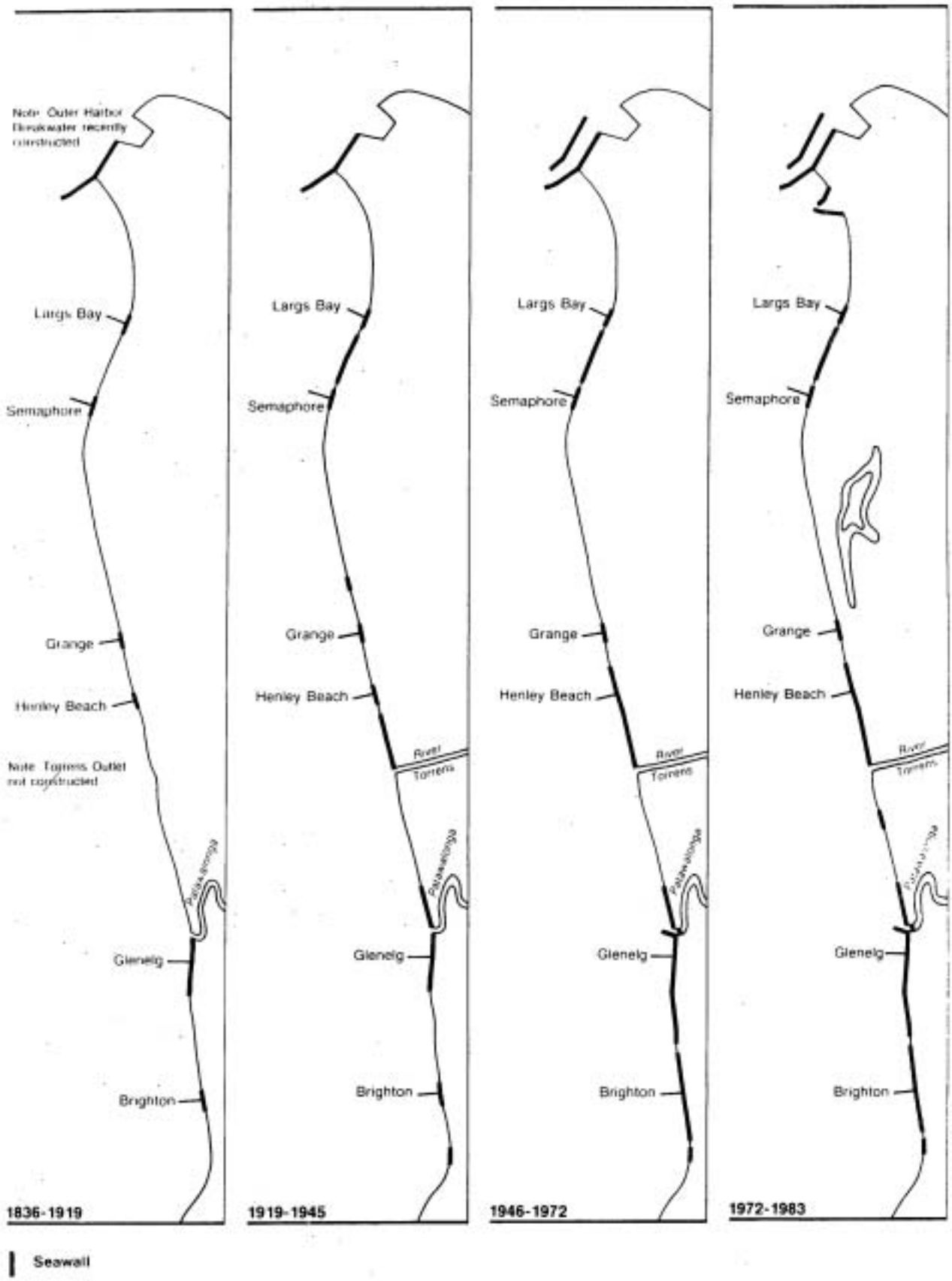
4.1.1 Coast Protection Works Before 1945

The earliest recorded protection works are an old stone pier built at Brighton in 1837, and seawalls constructed at Mosely Square, Glenelg, between 1856 and 1860. Other early work was mainly associated with construction of the jetties – to provide protection at their landward ends. The Semaphore jetty and the original Glenelg jetty were built in 1859, and the others between 1880 and 1883. Other early work included a timber wall at the Brighton jetty in 1915–17, and reconstruction and extension of the aforementioned seawalls near Mosely Square.

Coast protection needs increased as the coast became more developed between 1920 and 1945. The following seawalls were constructed during this period (the asterisk denotes structures that are still in place and that still present the front line of hard defence):

- concrete seawalls at the Brighton jetty (1926)
- the Rotunda at Wheatland Street, Seacliff* (1930 approximately)
- 1200 feet of concrete seawall between the Patawalonga and Margaret Street, North Glenelg (1925–26)
- 700 feet of concrete seawall at The Broadway, South Glenelg (1927–28)

FIGURE 49 HISTORICAL SEAWALL CONSTRUCTION



- concrete retaining walls at the Henley jetty (1929) and Grange jetty* (1926)
- the Henley swimming pool with its seaward concrete wall (1933)
- a concrete seawall at Terminus Street, Grange* (1939)
- a timber piled concrete and stone wall between Ozone Street and Henley Beach Road, Henley Beach South* (1944–45)
- a concrete retaining wall north of the Semaphore Surf Life Saving Club* (1920s).

Those structures not asterisked were subsequently damaged in storms and have since been replaced. Damage that occurred during the major storms of 1946, 1948 and 1953 was partly due to the severity of these events, and partly because of the beach drop caused by the vertical, highly wave reflective structures. Following this damage and following further development of the coast, there was a need for increased seawall construction.

4.1.2 Coast Protection Works 1945–72

The period after World War II saw the first coordinated attempts at foreshore protection, with the establishment of special Local Government committees and their interaction with State Government. These committees were established to consider storm repairs following the major storms.

The Seaside Councils Committee, established after the 1953 storms, continued until the Coast Protection Board was established. During this period, the former Harbours Board seems to have played a more active role in providing coast protection advice and designs for protection work.

The following coast protection structures were built between 1945 and 1972 (as before, the asterisks indicate structures that are still in place):

- timber piling at Portland Street, Brighton, and road repair and protection at Somerton using railway sleepers (1948)
- timber sheet piling at several places north of the Brighton jetty, and also at Seacliff (1953)
- dumped rock along virtually the entire foreshore length of Brighton* (1953–58). Portions of this are still intact, mainly because of the beach replenishment program.
- replacement of concrete seawalls at the Anzac Highway car park, Glenelg*, and southward extension of the concrete seawall at The Broadway* (1953)
- loose stone placed between the southern Glenelg boundary and The Broadway (1960)
- construction of the Patawalonga groyne* (1964–65)
- a gabion seawall constructed along the dunal frontage of the Glenelg Sewage Treatment Works (1969–72). An initial 350 feet near the outfalls was built in 1969, and this was followed by construction along the entire frontage in 1970. Extensive repairs were required to rectify storm damage during the short life of this wall. Separate protection for the outlet pipes was provided at about this time, using rocks, reno-mattresses and concrete. Gabions are special wire-mesh cubes (approximately 1 m x 1 m x 1 m) filled with small stone and wired together. Reno-mattresses are a flat version of these.
- reconstruction of the seawall at the Henley jetty* (started in 1952 and completed in 1954)
- construction of a similar vertical seawall from south of the Henley swimming pool to South Street, Henley Beach* (1953)
- construction of a rock wall from South Street to Henley Beach Road* (1953)
- dumping of loose rocks north of Marlborough Street, Henley Beach (1955)
- reconstruction of the concrete wall south of the Grange jetty* (1966)
- provision of rocks in front of the concrete walls both north and south of the Semaphore jetty* (1960s).

Engineering details of these works, including drawings, are included in the Branch's Technical Report No. 81/7 (Moulds 1981).

In the early years, and persisting into this period, coastal engineering was coloured by the prevailing attitude in the western world that man should dominate and subdue the hostile environment to serve his purposes. In the case of coastal erosion, the perceived enemy was confronted head-on, mainly with concrete walls, some vertical and some curved, to better reflect the waves back to the sea. This was done with a confidence that managed to survive failure after failure.

It was only in the 1960s that this attitude started to give way to the realisation that engineering design, particularly at the coast, must be in sympathy with the environment if it is to succeed in the longer term without unwanted side effects. Emphasis changed from vertical seawalls (which caused beaches to scour, leading to undermining of the walls) and from groynes (which so often had erosional side effects) to softer, wave absorbent solutions. These included energy dissipative seawalls of placed rock (as used locally) and special concrete shapes, but even these caused some beach scour and had to be designed to take account of this.

More recently, replacement of the natural beach has been seen to be the most harmless way to neutralise wave energy. There has simultaneously been a welcome trend to avoid development on seafront dunes. Other recent coast protection methods are discussed in Section 5.1.1.

4.2 RECENT SEAWALL CONSTRUCTION (1970–83)

The Culver report, which recommended beach replenishment, was prepared and considered in the context of an ongoing program of seawall construction. Although it did not recommend continuing the seawalls, this seems to have been taken for granted because of the urgent need to protect vulnerable property, and because it would be some time before investigations were complete into the report's proposal to pump sand ashore at Brighton. These investigations were not fruitful (refer to Section 2.2.4), and beach replenishment proceeded on a small scale by trucking.

Seawall construction since 1970 is shown in Figure 50. This work was all to 'rip-rap' designs approved initially by DMH and later by the Board. Limestone from the Linwood quarry was used in all cases. The first of these 'modern' seawalls was built at Kingston Park and Marino Rocks in 1972, jointly by the Brighton and Marion councils under the supervision of DMH. This was followed in 1973 by reconstruction of portions of the Brighton esplanade protection (at the jetty, and between Downing Road and Gladstone Road). The design for this and for similar work carried out shortly after at North Glenelg (from the Patawalonga Outlet to the sewage treatment works) was done by consultants, and approved by the Board. The design followed that previously recommended by DMH. Subsequent northward extension of protection at North Glenelg (by the Engineering and Water Supply Department to protect the treatment works) was to a similar design recommended by the Board. A small section of similar walling was built at Chetwynd Street, West Beach (immediately north of the West Beach Recreation Reserve), at about the same time.

The early rip-rap design was very similar to the present one. It comprised a 2 m thickness of rock over a stone filter. The stone filter was of two layers, gravel of 3–65 mm and stone of 150 mm, chosen to prevent the fine material in the bank (mostly dune sand) washing out through the rocks. The outer armour–stone layer was of rock between 1 and 3 tonnes in weight. The main difference between the early design and the current one is that the early one did not include a special toe-stone. The earlier and more recent designs are shown in Figure 51. Since 1975 the design has included a row of 5 tonne toe-stones to help prevent undermining by containing the bottom end of the structure and especially the filter, which was vulnerable in the early seawalls.

Seawalls built since 1978 have had a slightly higher top rock level, by 0.6 m. The level was raised to reduce minor scour, which was occurring above the rocks. This was probably due to the beach levels falling and larger waves consequently reaching the rocks, and causing more overtopping.

FIGURE 50 SEAWALL CONSTRUCTION SINCE 1970

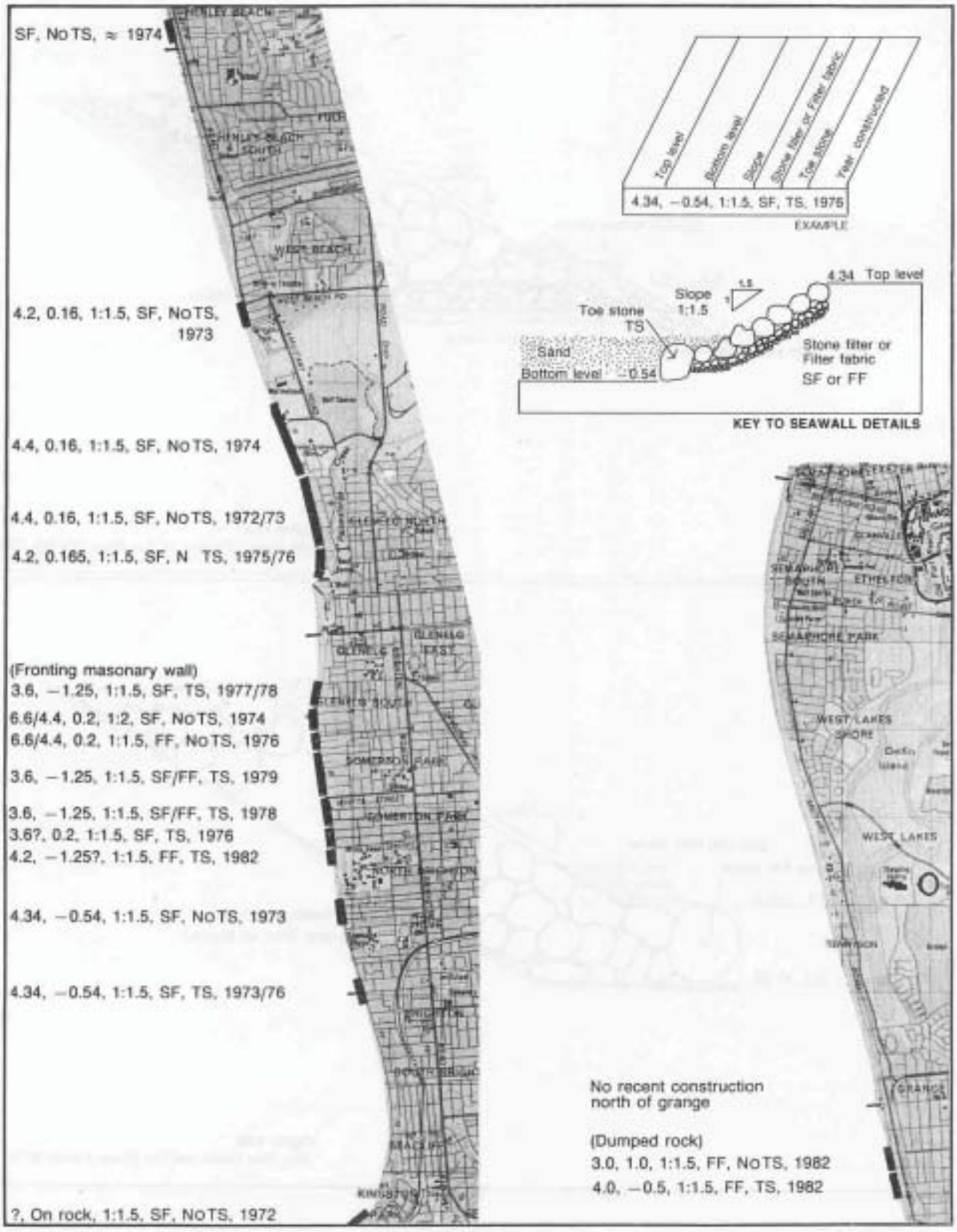


FIGURE 51 RIP-RAP SEAWALL CONSTRUCTION

Figure 51A
With Stone Filter and No Toe Stone (Before 1975)

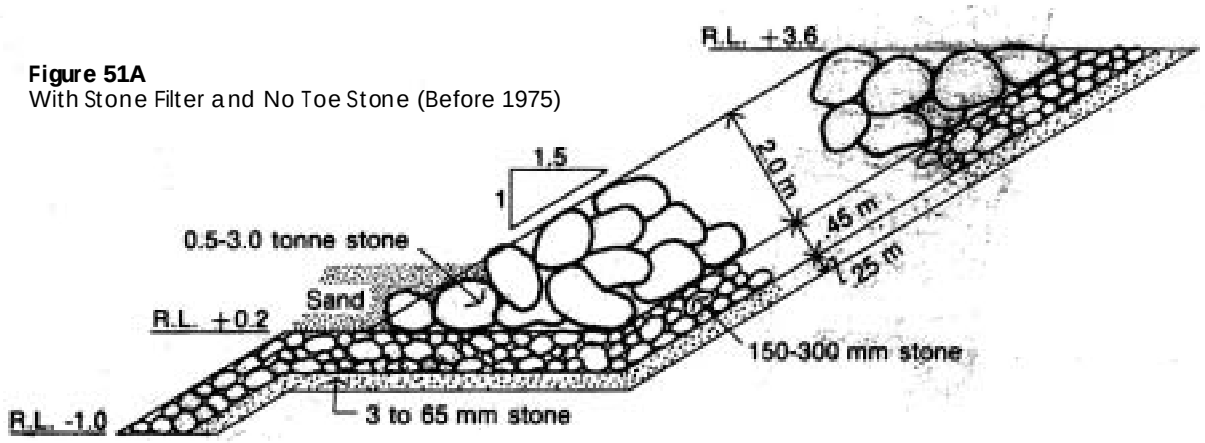
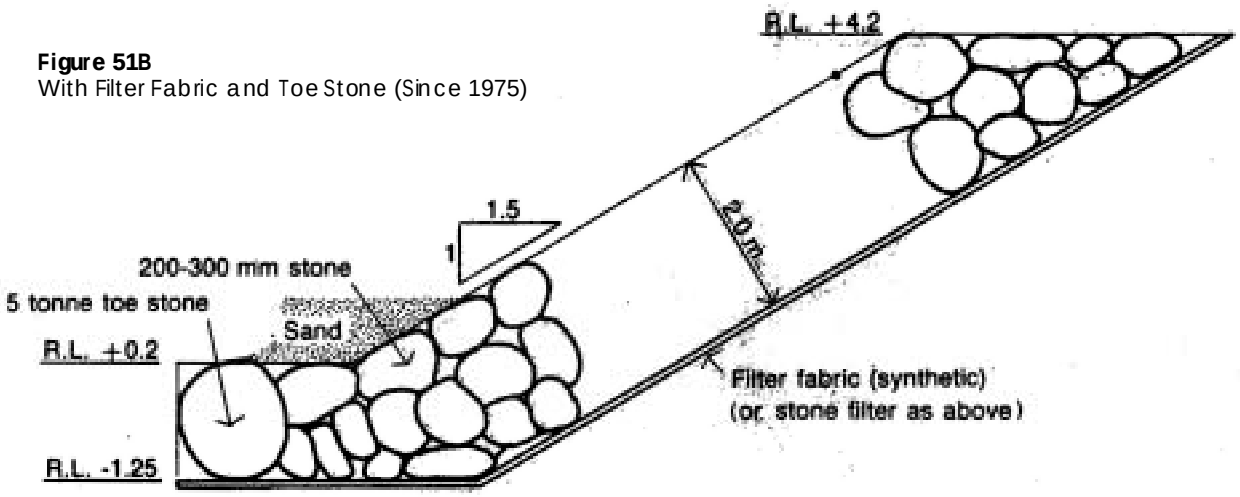


Figure 51B
With Filter Fabric and Toe Stone (Since 1975)



Reconstruction of seawalls continued in the North Brighton and South Glenelg areas between 1975 and 1980, including reinforcing the stone wall between The Broadway and the Glenelg jetty. This wall was provided with a rock face and toe, incorporating a filter as for the other new seawalls.

By 1980 the length of coast between Minda and The Broadway had been provided with new seawalls. South of Minda only a short length, Gladstone Road to Holder Road and at the jetty, had been reconstructed. The Board has recommended rebuilding the portion between the Brighton jetty and Holder Road, but this has not yet been done, and the need is now less certain because of the unexpected lack of damage (no doubt mainly due to the effects of beach replenishment). The dumped rock south of Bindarra Street (first street south of the jetty) to Seacliff has sufficed because of the beach replenishment, and there are no plans to replace it with a constructed seawall.

Two severe storms in winter 1981 required emergency rock to be placed on the beach at Somerton (in front of the Life Saving Club at the northern end of the Minda property) and at Henley between Marlborough Street and Grange Road. Both these areas were subsequently provided with new rock protection. The short length of new work at Somerton was to have been built to the standard design (including a filter fabric in place of the filter stones – see later), but the toe was placed at too high a level, because of on-site difficulties. This was corrected by placing an additional toe in front of the original, though still at a slightly higher level than called for in the design.

PHOTOGRAPHS RIP-RAP ROCK SEAWALL CONSTRUCTION

A compromise design was used for the 1981–82 work at Henley Beach, to reduce costs and to minimise the erosional 'end-effect' to the north. A 300 m length extending north from Marlborough Street was constructed to the full rip-rap design, using placed rock over a filter fabric, and the remaining 400 m to Grange Road was done with 'dumped rock', again over a filter fabric. The latter section was actually partly constructed (as compared to dumping) with some grading from small stone against the filter to the largest on the face, and would perform better than the specified dumped rock. It was set within the original dune, which was replaced in front of the rock after construction, and which is supplemented with replenishment sand each summer. The intention is that the dune provides the main defence for the northern part, with the rock behind providing emergency protection in the event of a major storm. Property would be protected, but some damage to the rock wall would be expected. No major storms have occurred since this seawall was built, and the dunal barrier has performed well for the minor events to date.

As mentioned, special synthetic filter fabric has replaced the stone filter in recent seawalls. The fabric has a cost advantage, as it simplifies the construction, and has so far proved at least as effective and possibly better than the stone filters. Failure of the stone filters has been the cause of most of the seawall maintenance to date. This has caused some slumping of portions of the seawall at North Glenelg and in front of the treatment works, as sand has washed out from behind. Further slumping, with consequent repair needs, can be expected in these areas. This problem has not occurred elsewhere, though there has been some nuisance caused by filter stones escaping onto the beach. Although filter fabrics are now used in a wide variety of soil engineering applications, use behind seawalls is particularly arduous. Because of this, only heavy-duty fabrics with high puncture resistance (to avoid damage during construction) and high abrasion resistance have been used. Some long-term deterioration may occur due to exposure to ultraviolet light, although the filter is well screened by the graded rock construction and deterioration should be very slow.

Arrangements for constructing and financing seawalls have altered considerably, even since the Board's establishment. Before the Coast Protection Act, they were built by councils, with and without Government grants. Since the Act was passed, such projects have been eligible for Government grants through the Board.

In 1973 work at Glenelg was started by a contractor (employed by the council), but was subsequently completed by the Government, using an Engineering and Water Supply Department construction team. Both this and the rip-rap, which the Engineering and Water Supply Department built at Brighton in 1973, were subsidised by the Board at a 662/370 level. The seawall fronting the Glenelg treatment works was carried out and mainly paid for by the Engineering and Water Supply Department. The Board contributed towards that portion not directly related to protection of the Engineering and Water Supply Department property.

Between then and 1980 an Engineering and Water Supply Department team was used exclusively, with costs being charged to the Board, which then recovered a one-third share from the council. In practice, this third was usually related to the estimated costs rather than the actual costs, which were generally higher.

The two 1981 seawalls were built as council projects. The Somerton one was designed and constructed by the council. The Henley one was designed and supervised by consulting engineers for the council and was built by a contractor with the Branch closely involved in checking the work. The Somerton work, being of more of a 'storm damage' nature, was fully funded by the Board. The main, protection part of the Henley project was funded at slightly less than 80%.

The only seawall work done since then has been a temporary strengthening of the dumped rock between the Henley jetty and the new work at Marlborough Street. The Board provided a grant towards the placing of large rock to provide temporary protection until funds are available for reconstruction. This work was carried out by the council in autumn 1983 to avoid winter storms causing damage to the road.

The present policy, which is consistent with the provisions of the Coast Protection Act, is that seawalls be considered as council projects, and that a grant of 80% will normally be applied regardless of whether the work is classed as new protection or storm damage repair.

4.3 THE BEACH REPLENISHMENT PROGRAM

The Board's beach replenishment program started in 1973–74 with the Brighton beach being replenished with 14,335 m^3 of sand from Taperoo, and with 24,777 m^3 being carted to North Glenelg, partly from Taperoo and partly from south of the Glenelg groyne. It continued on a small scale until 1977, when it was increased to 105,369 m^3 for the 1977–78 financial year. It has averaged 105,000 m^3 for the 6 years between 1977 and 1983. A history of the program is provided in a Branch Technical Report (Penney 1980). The program and its effects on the beach sediment budget have already been discussed in Section 2.6.3. A full description of the quantities carted, the sand sources, and the replenishment beaches is provided in Figure 34.

While emphasis has been on counteracting the northwards littoral drift by carting sand from Semaphore to Brighton, the program has included replenishment of other problem areas, such as North Glenelg and West Beach, and sand has been obtained from other areas of accumulation, particularly where it has become a nuisance. Sand has been obtained from within the study area at Taperoo, Largs Bay, Semaphore (both south and north of the jetty), Port Malcolm, Grange (at the jetty), south of the Torrens Outlet, and from south of the groyne at Glenelg. As opportunities have arisen, sand has occasionally been obtained from coastal building sites, and from Port Stanvac where it has been dredged ashore to counter silting. The latter has been a joint arrangement, with the refinery paying the loading costs.

Semaphore and Glenelg (south of the groyne) have been the main sources. Most of the sand has gone to Brighton, though the proportion to North Glenelg has been increased over the past few years in an attempt to prevent the rapid loss of beach from this area, and to retard erosion of the West Beach dunes. Between 1977 and 1980 the beach fronting these dunes was replenished directly by transporting sand along the beach by scraper from south of the Torrens Outlet. However, the effects were temporary and too limited. Increased replenishment at North Glenelg has been found to be at least as effective, and to benefit a larger area. It also avoids the hazard of heavy scrapers moving along the beach.

Sand was initially taken from south of the Glenelg groyne in attempts to reduce silting of the boating channel, and this source has continued to be used, partly for this reason, and partly because it is easily accessible. The extent to which this has helped the boating channel is uncertain, but the effect has probably been small. Sand movements around the groyne and across the channel are not well understood (refer to Section 2.6.3). Some natural bypassing is almost certainly occurring. Removal of sand from the south might reduce this (thus contributing to the erosion problem to the north), but the effect is probably slight. The outlet could be artificially bypassed by taking sand from the south and placing it on the north Glenelg beach, but this is inefficient because it costs relatively more to load the sand and carry it only a short distance, and because moving sand northwards adds to the quantity that must eventually be returned to the south. The strategy has therefore been to replenish North Glenelg from Semaphore or the Torrens Outlet, and to use the South Glenelg sand for Brighton and Seacliff. This is in addition to the main strategy of taking sand from Semaphore to Brighton.

The large source of accumulated sand south of the Torrens Outlet has not been used to the extent that it perhaps could have been, because of a concern that removal of sand could adversely effect the erosion-prone coastline at Henley to the north. This is discussed in Section 2.6.3, where it was concluded that the concern is unwarranted.

A recent inclusion in annual programs (since 1981) has been the placing of 15,000 to 20,000 m^3 of sand to maintain the beach and dune in front of the dumped rock at Grange Road and to prevent an erosional end-effect from this new seawall.

PHOTOGRAPHS BEACH REPLENISHMENT

Recent analysis of beach and replenishment sand grain size has indicated that beach sand from Largs Bay northwards is too fine for cost-efficient replenishment, and that this applies also to the sand that had previously been obtained from Port Stanvac (refer to sections 2.2.4 and 2.6.3). A Branch study (Fotheringham 1982) showed that the sand from both south and north of the breakwater at Port Stanvac might be too fine, especially the dredged material on the north. Re-evaluation of the data shows that the larger deposit to the south is suitable, and this will be used in the future. Removal of this sand will benefit Port Stanvac by reducing the rate of silting, though the benefit will be less direct than the previous removal of the dredged material. Fotheringham showed that removal of sand from Port Stanvac was unlikely to adversely affect beaches in the Noarlunga area, a concern that the Noarlunga Council had expressed.

The beach replenishment program has been arranged and paid for entirely by the Board, though close liaison has been maintained with councils, with agreement being reached (sometimes with difficulty) on source beaches, truck routes and other details.

The program has not been without difficulties, the main ones being the need to run trucks in residential streets, sand on roads (especially where the trucks come on and off the beaches), the need for trucks and machinery to operate in a recreational area (with some attendant risk), and some temporary adverse effects on the source beaches. These problems have persisted and attracted public complaint despite considerable efforts to alleviate them and despite prompt attention to all complaints. These environmental effects are discussed in the comparison of alternative strategies (Section 5.9).

Another problem has been the difficulty in obtaining truck access to beaches, and in moving enough sand to reduce the interaction of the sea with the seawalls. Because of access difficulties, replenishment at Brighton has all been south of the jetty and, as discussed in Section 2.6.3, the benefit north of the jetty has been limited. Erosion effects at Somerton may be partly due to the small amount of sand and the way the seawalls affect its transport. A new ramp has recently been built to allow access to the beach north of the Brighton jetty. This will alleviate some of the problem. However, replenishment should ideally be in greater quantities, and should be more evenly spread as far north as The Broadway. This latter area presents a problem because of the lack of beach access and because it would be difficult to avoid the use of residential streets.

Notwithstanding these difficulties, the program has proved effective in countering the effects of erosion – as shown by the small amount of damage sustained in recent years, despite severe storms in 1981. It has also done much to preserve recreational beaches. Although the seawalls have caused some loss of beach, this loss would have been far greater had the beach replenishment program not been implemented.

The northern beach sand reserves are unfortunately limited and this might preclude long-term extension of the present methods. Sand quantities on the source beaches and in other possible reserves are discussed in Section 5.4, and extension of the present policies in Section 5.7.2.

4.4 DUNE STABILISATION

A program of dune stabilisation has been combined with the beach replenishment, to prevent loss of sand inland and the nuisance associated with this. This has involved the erection of sand drift fencing (and its replacement when washed away by storms) and some planting. The Woodville Council has used these methods to maintain a considerable length of coastline, and work by the Henley and Grange Council at the Torrens Outlet, and more recently at the new Grange Road seawall, has also served its purpose well.

Sand drift problems have persisted at South Brighton, behind the replenished beach, and sand has needed to be periodically cleaned off the esplanade and returned to the beach – despite the construction of drift control fences. A more concerted effort in early 1983, involving the erecting of more extensive fences and the use of tree prunings on the small dunes, has proved effective.

This dune stabilising work has been carried out by councils, generally with an 80% grant from the Board.

Inland loss of sand at the West Beach dunes has continued to be a problem, and the Board has put considerable money and effort into stabilising these dunes. This portion of the coast is managed by the West Beach Trust, which, unlike a council, is not eligible for grants under the Coast Protection Act. In the consequent uncertainty about financial responsibility for the dunes, the Branch has twice arranged major rehabilitation, between 1976 and 1979 and in 1983. This has included reshaping the dunes with earthmoving equipment, installation of a sprinkler system (to use effluent from the nearby sewage treatment works), fencing, planting, and the provision of walkways to the beach. This work has been fully paid for by the Government, through the Board. By agreement between Ministers, regular maintenance will now be carried out by the Trust, with the Board paying for special work resulting from storms. Legislative amendment, to enable the Board to provide grants to the Trust, is under consideration.

Local techniques for dune management are now well established, and sand drift need only be a problem if routine maintenance and storm repairs are neglected.

CHAPTER 5: ALTERNATIVE STRATEGIES

5.1 INTRODUCTION TO ALTERNATIVES

It is worth a reminder at this stage that the present coast protection strategy – annual beach replenishment combined with a small program of seawall construction – is not that recommended in the 1970 University of Adelaide report. That report recommended beach replenishment, and although quantities were not specified, those used for calculating approximate costs were considerably less than the replenishment quantities considered in this review. It is also worth noting again that the present strategy has provided effective protection and some much improved beach at a reasonable cost. It performed well for the two severe storms in 1981, though has not been tested by a sequence of stormy years.

The questions that this review needed to address were:

1. How significant is the problem, and are continued protection measures warranted? If so, how much expenditure is justifiable?
2. How practical are various possible alternative strategies and how do these compare with the present policy? This needed to be considered from cost, environmental and social viewpoints.

These questions, set out in more detail, formed the brief for the Alternatives Study, which was carried out for the Board by the consortium of Kinhill Stearns and Riedel & Byrne. This chapter consequently draws heavily on the consultants' findings, but also includes other material, and is not in agreement with the consultants' findings on all points. It should be noted that the Alternatives Study was carried out in close liaison with the Branch, and that differences are not large, and are mainly based on additional work done in the Branch.

The first of the two questions has already been answered in preceding chapters. The erosion problem is due to natural processes that cannot be altered. The processes will continue and, if no further protective action is taken, will result in millions of dollars of property damage and the loss of much of Adelaide's recreational beach, also valued in millions of dollars. Further protection measures are clearly warranted, and expenditures of the order of those considered in this review are shown to be justifiable.

The second question is the subject of this chapter. The environmental and social aspects are dealt with in Section 5.9, which follows consideration of the alternatives but precedes their comparison. Material is drawn from this where appropriate for inclusion on the discussion of each alternative. This enables all the important aspects of each alternative to be considered together.

It was considered important that the range of alternatives be as wide as possible, and that the review 'put to rest' impossible or impractical options as well as compare the practical ones. To avoid the impractical options causing unnecessary confusion, they are discussed separately in Section 5.1.2, and thereafter ignored.

As will become evident, the practicality and costs of dredging sand from offshore have an important bearing on the review. Quite detailed information, such as the location and depth of offshore sand deposits in relation to types of dredging plants and the availability of such plants, is very significant. This is therefore dealt with separately in Section 5.2, before the main discussion of alternatives.

It is useful to first consider the various protection philosophies in relation to local coastal processes. This will provide a better understanding of the discussion on each alternative and will help explain why some methods can be dismissed.

5.1.1 Coast Protection Philosophies

There are only three fundamental coast protection strategies, and these include all the alternatives considered in this review. They are:

1. To move development back from the coastline – following which the coastal processes continue but cease to be a problem for the time being.
2. To alter the rate of erosion of the coast by supplying erosion material (beach nourishment), by building seawalls, or by reducing wave energy (breakwaters).
3. To manipulate the alongshore transport process (where this exists) to obtain selective accumulation at some places and, hopefully, predictable and acceptable erosion at others (groynes, offshore breakwaters, and artificial headlands).

The last two basic strategies obviously cannot be entirely separated, because anything that affects wave energy must also change the alongshore transport that is driven by this energy. Some of the alternatives have elements of both these basic approaches.

Relocating Development

The option of relocating development back from the coast is rarely viable for social reasons and, for intensely developed coastlines, cannot be justified when costed over normal planning periods. However, it may well be the least costly option when considered over one or two hundred years. This option was not included in the Alternatives Study, though the study's property loss estimates give an idea of the order of costs that a relocation strategy would involve. The costs of relocation would be considerably higher than the 'no action' losses discussed in Section 3.1.2.

Given the existence of the seawalls, a relocation strategy would do nothing to prevent loss of beaches and, given the demonstrated value of these (Section 3.4), can be dismissed for this reason alone.

Another possibility could be to relocate property and to remove the seawalls. This would enable the beach to be retained, but could only be achieved at very high financial and social cost. A wide strip of property would need to be acquired to allow for the initial beach adjustment as well as for long-term erosion.

A compromise on relocation, which could be used in conjunction with a protection strategy, would be to minimise the value of property at risk, by zoning to avoid more intensive development or by Government purchase as property came onto the market. The latter would assist in keeping future options open. Such restrictions on property development are unlikely, however, to be necessary at Adelaide within the foreseeable future, given the economic practicality of the protection measures considered in this report, the buffer space between most private property and the seawalls, and the limited intensity of even the most extreme local storms. However, this situation could change if a rapid sea level rise should occur (see sections 2.3.6 and 5.10.7).

Altering the Rate of Erosion (Seawalls)

This approach on its own is only applicable where there is no alongshore transport and where erosion is occurring as a result of loss of sediment to offshore areas. This is hypothetical here (because alongshore transport is an underlying factor), but a 'seawalls only' strategy fits this category. Beach replenishment also does insofar as it provides a storm or erosion buffer, though it can also be considered as a soft, non-structural way of manipulating the alongshore transport – when the sand source is from within the particular beach system.

One of the problems of seawalls is that they are essentially rigid structures, which, on a sandy beach, separate elements of the ever changing dune–beach–bar system. During a storm, the waves first move the beach material onto the offshore bar and then reinstate the beach with material eroded from the foredune. On an undeveloped coast this is reversed during calm conditions, when the lower, longer-period waves return the sand from the bar to the beach, and when onshore winds move dry beach sand back to the dune face, where it is trapped by

vegetation. When a seawall is introduced in the middle of this system, the beach drops further and some material is lost offshore to depths from which it may not be returned; or it may be lost alongshore due to an increased transport caused by the extra turbulence. The result will be progressive loss of beaches until the seawall fails through being undermined or until a hard layer is reached and the beach above has been lost.

Providing a buffer by beach replenishment does not have these disadvantages, mainly because it complements the natural processes instead of interfering with them as seawalls do. The natural seasonal beach processes continue, but dune erosion, which is the usual threat to property, is avoided because of the wider beaches and the new foredunes.

Replenishment is often used (as in the study area) when beach loss caused by seawalls becomes unacceptable and threatens the stability of the walls. However, part of the benefit of the replenishment is lost if beaches are still too low to prevent normal tides and minor storm tides from reaching the seawalls – these then continue to interfere with the natural processes, causing more rapid loss of the new beaches. Providing seawalls and sufficient sand to prevent the sea reaching the walls (except possibly during major storm events) is obviously a costly 'belt and braces' approach. However, it can be economical if the seawalls are designed to provide emergency protection only, and possibly to fail themselves while protecting property behind. This strategy was used for the northern part of the most recent seawall built at Henley Beach.

Using replenishment as a buffer instead of seawalls avoids the scour that occurs at the end of lengths of protection. This was the reason for adopting the seawall/replenishment strategy for Henley Beach. If a seawall had been used alone, or with only a small replenishment, erosion would have occurred to the north, and the seawall would soon have had to be extended.

Reducing wave energy, possibly by partial but continuous breakwaters, has the main effect of reducing the seasonal beach change. It also reduces alongshore transport in proportion to the energy reduction (and possibly also by causing the waves to refract more and to approach the coast more directly). However, for practical purposes, some alongshore transport would remain and, in the local situation, sand would continue to move northwards from Brighton. Discontinuous reduction of wave energy causes changes in the alongshore transport, and falls into the third philosophical category.

Modifying Alongshore Transport

Modifying the alongshore transport with groynes or breakwaters is invariably more risky than other strategies, especially where the direction, distribution and quantity of alongshore transport are not well established. It is usually the least predictable and the most damaging action that can be taken. However, these methods can provide practical and cost-effective protection in the right circumstances. These are generally when there is an adequate natural or artificial supply of sand, where wave energy can be reliably predicted, and where downdrift erosion can be tolerated.

These methods, which were once very popular, have an unfortunate history of failure, either by failing to produce expected sand build-up or by producing unpredicted and unacceptable erosion. The idea of a one-off structural solution is, however, understandably attractive, especially to those who are better acquainted with the historic use of groynes than with their failures.

The need for caution is especially relevant at Adelaide where there is an unusually high seasonal variation in wind and wave direction and in alongshore transport. As discussed in sections 2.5 and 2.6.3, the average annual alongshore transport has not been accurately quantified, though it is known to vary considerably along the length of the study area as well as from year to year. In these circumstances, strategies that rely on modifying the transport are inherently suspect and must be treated very cautiously.

Also relevant to this is the virtual absence of sediment transport into the study area. This means that any modification of alongshore transport will cause a redistribution of the present limited amount of sand. More sand could be achieved at some places, but only at the expense of others. At Adelaide, a structural solution to modify the transport would cause more erosion, unless it were combined with adding more sand to the beaches.

Structural solutions not only cause build-up of sand on the updrift sides of the structures, but also cause the new beach alignment to be more closely normal to the average incoming wave energy. This results in reduced alongshore transport, or in this being totally stopped. This type of solution, if practical, would reduce or remove the need for top-up replenishment to the southern beaches.

A consequence of retarding the rate of transport along a portion of a coastline is that the downdrift coastline is deprived of this material and, assuming that it is also subject to alongshore drift in the same direction, will erode. A sacrificial length of coast is therefore often needed. It may sometimes be possible to use this to advantage by arranging for this deficit to occur where useful – eg at a harbour entrance (to reduce dredging) or to keep a major drain open. In the Adelaide situation, the only sacrificial areas are at North Haven and Outer Harbor, where a sand deficit would be welcome. However, this is too far north to be of much use to a strategy aimed mainly at the southern part of the study area. A possible dredging saving at the Patawalonga (see Section 5.5.2) is included in the comparison of alternatives.

Another point that is often overlooked is that those structures that cause realignment of beaches can increase the effective length of the coast, and that more sand may be needed if the average beach width is to be maintained. This should be allowed for to cater for storms from directions other than the average. At Adelaide, storms from the north-west are quite common, despite the dominant wave direction being from the south-west and major storms coming from that quarter.

Beach replenishment is a form of manipulation of the alongshore transport when the sand source is from beaches within the immediate system – as applies for the present sand carting program. Source beaches are deprived by the amount of sand taken, while alongshore transport into and out of the source area remains virtually unchanged. This has been acceptable at Adelaide so far, because of unwanted sand accumulation on the northern source beaches. Estimates of these reserves (Section 5.4) suggest that they will be exhausted in approximately 15 years, after which unacceptable erosion could occur.

The earlier comments in this section, about the advantage of beach replenishment over structures (because it has less interference with the natural processes), apply to structures in this category as well as to seawalls.

Assuming that replenishment does not alter the alongshore transport rates, sand from an offshore source would not affect the downdrift coast except through a temporary spreading effect. (Providing there is sand on the beaches, it will move northward at approximately the same rates, regardless of the width of the beach.) The effects that either a major replenishment or the lack of beach sand may have on the alongshore transport rates are discussed in Section 2.6.3.

The particular relevance of the remarks in this section is discussed for each alternative under the separate headings 5.7.1 to 5.7.8.

5.1.2 Strategies That Can Be Dismissed

The **'no action'** and **relocation** options have already been mentioned and the consequences of taking no action have been discussed in Chapter 4. The property and beach losses identified in the Alternatives Study clearly show that the taking of no action cannot seriously be considered as an option. Discussion in this chapter has already noted that property relocation deserves even less consideration – unless an unusually long, and economically unjustifiable, planning period (100 years or more) is adopted.

Other alternatives that the consultants dismissed were floating breakwaters, artificial seaweed, shaped offshore dredging, and groynes (or other structures to modify alongshore transport) unless combined with beach replenishment.

The **groynes without beach replenishment** option was dismissed because of the limited sand supply from the south and the short supply of sand on the beaches.

Floating breakwaters are only useful for small waves of short wavelength, and were dismissed as being impractical and ineffectual in the local wave climate. They have also generally not proved economical in practice, mainly because of their high maintenance requirements.

Natural seagrasses undoubtedly modify beach processes by reducing wave energy (see discussion in Section 2.7), and **artificial seaweed** has a similar effect though this may be slight. As discussed, this effect is mainly in reducing the seasonal on/off beach sand movement. Either real or artificial seagrass would have a lesser effect on the alongshore transport, which underlies the local problem. The consultants considered that the technology of artificial seaweeds was not sufficiently advanced. There is as yet no convincing evidence of coast protection being adequately achieved by this method. Although replanting seagrass in areas from which it has been lost (mainly between Glenelg and Grange) may deserve further consideration after methods have been developed, this does not offer a solution to the problem areas at Brighton, where the natural seagrasses have been stable and seem to have reached a natural equilibrium.

Offshore seabed contours can be deliberately altered by dredging in order to change wave refraction patterns and alter the distribution of wave energy reaching the coast. This method is only suitable for short lengths of coast, and could not be applied usefully here.

5.1.3 The Cost Comparison Method

Present and future costs need to be brought to a common basis so that they can be added and compared for each alternative. This is done by applying a 'discount rate' to future expenditures to obtain a present day value of the future expenditure. The discount rate is essentially an estimate of the difference between interest rates and inflation. It is used to calculate a hypothetical sum of money, which, if invested today, would provide the required amount to meet ongoing project costs (in real purchasing terms at the particular time in the future). Since the discount rate takes inflation into account, cost estimates for future items can be made using today's money value.

The main problem is that the future discount rate must be estimated, and is by no means certain. In practice, the difference between the interest rate on government borrowing and inflation can vary widely over even a few months. It is therefore desirable to do the cost calculations for a range of possible discount rates to ascertain the sensitivity of the cost comparisons to the rate used. The consultants selected rates of 2%, 5% and 10%, these giving a good range on both sides of the 1982 rate applicable to government borrowing. The sensitivity of the comparison to the different rates is discussed in Section 5.9. As would be expected, the higher discount rates favour those alternatives that involve later expenditure, giving relatively lower present day values for these.

The choice of discount rate is also a measure of the extent to which costs of projects that may mainly benefit future generations are borne today (too low a discount rate), or vice-versa, a measure of the extent to which present costs are minimised by passing costs onto future generations (too high a discount rate). The latter can be a deliberate strategy when technological advances can be anticipated, enabling future generations to solve the problems at less cost. However, the strategies here all involve substantial energy and transport costs, which seem unlikely to be reduced through foreseeable technological advances. A high discount rate does not seem to be justified on these grounds.

The discount rates considered for cost comparisons in this review are lower than those that might be considered for a private enterprise project. This is because private enterprise must see a higher future return (equivalent to a lower present value) to cover taxation.

In commenting on the Alternatives Study, Treasury has confirmed that the 5% rate is the most suitable, and suggested that comparative costs also be calculated at 7.5%.

The discount rates have been applied in slightly different ways here and in the Alternatives Study. Calculations here are based on costs being incurred in the middle of each year and being discounted to and summed at the middle of the first year. The Alternatives Study assumed that costs were incurred at the end of each year, and they were discounted and summed at the start of the first year. Both methods are acceptable and serve the present purpose, though neither is

strictly accurate, because lead times, which would be needed for the major projects, are not taken into account. The methods give different values by up to $1.5m. Cost comparisons should therefore not be made across the two reports, though they are valid within each. No adjustment has been made for cost rises above the 1982–83 values assumed in the Alternatives Study.

The discount rate is applicable to future savings and losses as well as to expenditures, and has been applied to all these equally.

Three warnings need to be given lest too much weight be placed on this type of cost comparison. Firstly, the present day values are only as good as the estimates of future expenditures and their timing. Even the best estimates are uncertain when the work is to be done in the quite distant future, and the time estimate is even more prone to uncertainty. Secondly, the method quite correctly gives very low present values for expenditures after say 20 years. This takes much of the meaning out of comparisons where large costs are incurred between the 20th year, for example, and the end of the 50-year period used in this review. While this is strictly correct and appropriate, it needs to be borne in mind. It also encourages procrastination where this may not be justified for other reasons.

The third warning is perhaps the most important. This is that there is a tendency for the economic aspects (derived by this correct but rather inflexible method) to outweigh practical engineering factors. For example, in comparing options that involve either a large dredging contract or purchase of a smaller dredge (to spread the work into the future, thus reducing the present day cost), the economic comparison will favour the latter, though there may be very good practical reasons for choosing the former. It is important to consider all factors, and not only the comparison of present day costs.

It is also worth noting here that protection methods that are spread over a number of years not only have an economic advantage, but also allow greater flexibility. Given the uncertainties in all of the alternatives considered, this can be a considerable advantage. It also has the economic advantage that a program can be reduced if found to be providing an unnecessarily high standard of protection.

5.2 DREDGING METHODS FOR USE OF AN OFFSHORE SAND SOURCE

It is appropriate to consider dredging feasibility before the alternative protection methods. This is because several of the alternatives rely on beach replenishment from an offshore source, and the practicality and costs of this depend on quite detailed dredging questions. These are dealt with in the consultant Dredging Report, and as only a limited number of copies of this report are available, it is useful to provide a summary of the relevant dredging techniques and to discuss some important aspects.

As discussed in the introduction to this report, it was decided not to further investigate offshore sand reserves until this review had first shown their use to be a practical and economical possibility. This meant that the Dredging Study and Alternatives Study had to be based on one offshore source, which had been only partly established, and on two other sites that can still only be considered as possibilities. This review has since shown that replenishment from offshore is a practical and important option, and an offshore survey is now under way. Unfortunately, the results will not be available in time for inclusion in this report, but the possibilities are foreshadowed in its conclusions.

The Board commissioned for the Dredging Study a consortium of a local firm, Lange, Dames and Campbell, and Pickands, Mather and Co., a US firm having the Savage River Mines in Tasmania as its subsidiary. The latter brought to the study its expertise in transporting material long distances by pipeline. The consultant was briefed to examine all practical methods of winning the sand from the three sources and transporting it onto the beaches between Seacliff and North Glenelg. The consultants' report, 'Replenishment of Adelaide Beaches from Offshore Sand Sources – Feasibility Study, April 1982', is referred to here as the 'Dredging Study' for convenience.

TABLE 7 COMPARATIVE COSTS – USE OF OFFSHORE SAND FOR BEACH REPLENISHMENT

Key	Plant	Capacity	Duration Years	Borrow Area	Discounted Total Cost ($m)	Quantity In Place Millions m^3	Rate/m^3 In Place ($)	Quantity Shifted Millions m^3	Rate/m^3 Shifted ($)
1	Hopper dredge	4,000 m^3	1	C	13.6 (10.6)	3 (2.3)	4.5 (4.6)	3 (2.3)	4.5 (4.6)
2	Hopper dredge custom built	1,000 m^3	3 (2½)	C & B	13.7 (11.3)	3 (2.3)	4.6 (4.9)	3.4 (2.6)	4.03 (4.3)
3	Hopper dredge 'Thomas Hiley'	1,600 m^3	2 (1½)	C	15 (12.1)	3 (2.3)	5.0 (5.3)	3.0 (2.3)	5.0 (5.3)
4	Hopper dredge custom built	1,000 m^3	10	C	12.5	3	4.2	3	4.2
5	Hopper dredge custom built	1,000 m^3	3½ (3)	B	15.7 (12.9)	3 (2.3)	5.2 (5.6)	3.6 (2.8)	4.4 (4.6)
6	Hopper dredge custom built	1,000 m^3	10	B	13.1	3	4.4	3.6	3.6
7	Hopper dredge 'Thomas Hiley'	1,600 m^3	2	B	15.2 (13.2)	3 (2.3)	5.1 (5.7)	3.6 (2.8)	4.2 (4.7)
8	Cutter suction dredge/pipeline	240 m^3/hr	3½ (3)	B	16.5 (14.7)	3 (2.3)	5.5 (6.4)	3.6 (2.8)	4.6 (5.3)
8A	Cutter suction dredge/pipeline	240 m^3/hr	3½	A	17.9	3	6.0	3.6	5.0
9	Dump barges and bucket dredge	500 m^3	2½ (2)	A/B	19.6 (5.6)	3 (2.3)	6.5 (6.8)	3.6 (2.8)	5.4 (5.6)
9A	Dump barges and bucket dredge	500 m^3	10	A	19.5	3	6.5	3.6	5.4
10	Cutter suction dredge/trucks	–	10	B	17.1	3	5.7	3.6	4.8
11	Hopper/Cutter suction dredge	3,000 m^3	1	C	21.1 (16.8)	3 (2.3)	7.0 (7.3)	3 (2.3)	7.0 (7.3)
12	Cutter suction dredge/pipeline	80 m^3/hr	10	B	18.8	3	6.3	3.6	5.2
13	Cutter suction dredge/train	–	10	B	27.0	3	9.0	3.6	7.5
(The proposal below provides for dumping offshore)									
14	Bucket dredge Dump barges offshore	500 m^3	2 (1½)	A/B	16.6 (13.0)	3 (2.3) assuming no losses	5.5 (5.7)	3.6 (2.8)	4.6 (4.6)

Note:

1. Total costs refer to initial major replenishment costs only and do not include maintenance replenishment costs. The discount rate used was 5%.
2. Unit rates for sand in place do not reflect differences in sand losses that may be experienced between the long- and short-term replenishment options.
3. 'Hopper' and 'Hopper dredge' are abbreviations for Trailing Suction Hopper Dredge.

Source:

Lange, Dames and Campbell Australia Pty Ltd (1982). The figures have been adjusted to increase the short-term quantity to 3 million m^3. The original figures are shown in brackets. Methods 8A and 9A have been added.

The Department of Marine and Harbours has examined the report, and generally agreed with the findings and the recommendations, within the limits of the assumptions. It was not in a position to comment on these. The Engineer for Deepening, DMH, has provided valuable comment on certain aspects, and this is included in the discussion here where appropriate.

The consultant was required to consider 2.3 million m^3 of sand and two project time periods – rapid (1–3 years), and over 10 years. The consultants for the Alternatives Study revised the sand quantity to 3 million m^3, and made appropriate adjustments to the figures from the Dredging Study, using the same methods as the first consultant. The revised cost comparison (Table 7) sets out the adjusted cost estimates, which include further revisions and additions made by the Branch. The method of estimating present day values, as described in Section 5.1.3, was also used in the Dredging Study. Discussion here assumes a 5% discount rate.

5.2.1 Sources Assumed for the Dredging Study

Section 2.2.4 describes work carried out in search of offshore sources and in proving a borrow area in the sandbanks north of Outer Harbor. As discussed, it was decided to abandon efforts to find a more convenient deposit off Brighton, because of negative findings from earlier work. When the Dredging Study was commissioned, the two most likely possibilities, other than the marginally suitable Outer Harbor deposit, seemed to be the general area off North Haven, and off the Onkaparinga River.

The North Haven area was an obvious contender because sediment has accumulated there since the Outer Harbor breakwaters were built shortly after the turn of the century. Use of this area would also be advantageous because this would reduce future dredging needs at North Haven and in the Outer Harbor channel. Unfortunately, recent dredging to maintain the North Haven entrance has shown that the sand is much too fine to be used. Further testing of a larger area is needed before this source is entirely rejected.

The only evidence for a deposit off the Onkaparinga is seismic information (Hails and others 1983), which suggests the presence of old river channels that have been filled in with sediment – possible coarse material brought down by the river. In addition, this is the only place locally where surface sediment has been found to become coarser in a seaward direction.

The three sources assumed for the Dredging Study are shown in Figure 52.

The actual location of the deposits does not markedly affect the dredging cost estimates, unless there is a large difference in the distance from the replenishment beaches, and unless the water depth and exposure influences the choice of dredging plant. The dredging cost estimates are thus useful, regardless of whether or not the assumed deposits are the ones used. Costing is at this stage also only at a very preliminary feasibility level, and any definite proposal would need to be costed in more detail.

The depth of a deposit and its distance from shore are important because this influences the choice of dredging plant and can affect coastal processes. The larger trailer-suction hopper dredges require a minimum operating depth of approximately 6 m at low tide, and also a reasonably consistent deposit – the method is unsuited to selective dredging. By comparison, a small or medium dredge of this type, or a larger cutter-suction dredge, might have a range of working depths between 4 and 20 m.

Dredging nearshore can alter coastal processes and may cause erosion by changing wave refraction (which is caused by the waves' interaction with bottom contours). This could cause wave energy concentrations at the coast by a 'lens' effect. Although some effect is possible for sources anywhere in the gulf, because of its shallowness in relation to wave lengths, the effect is only likely to be significant when dredging is close to shore in depths less than approximately 10 m, and then only in special circumstances, depending on the shape of the dredged hole. An investigation of this effect was not appropriate for the review, but would need to be done once a specific site was decided.

FIGURE 52 OFFSHORE SAND SOURCES ASSUMED

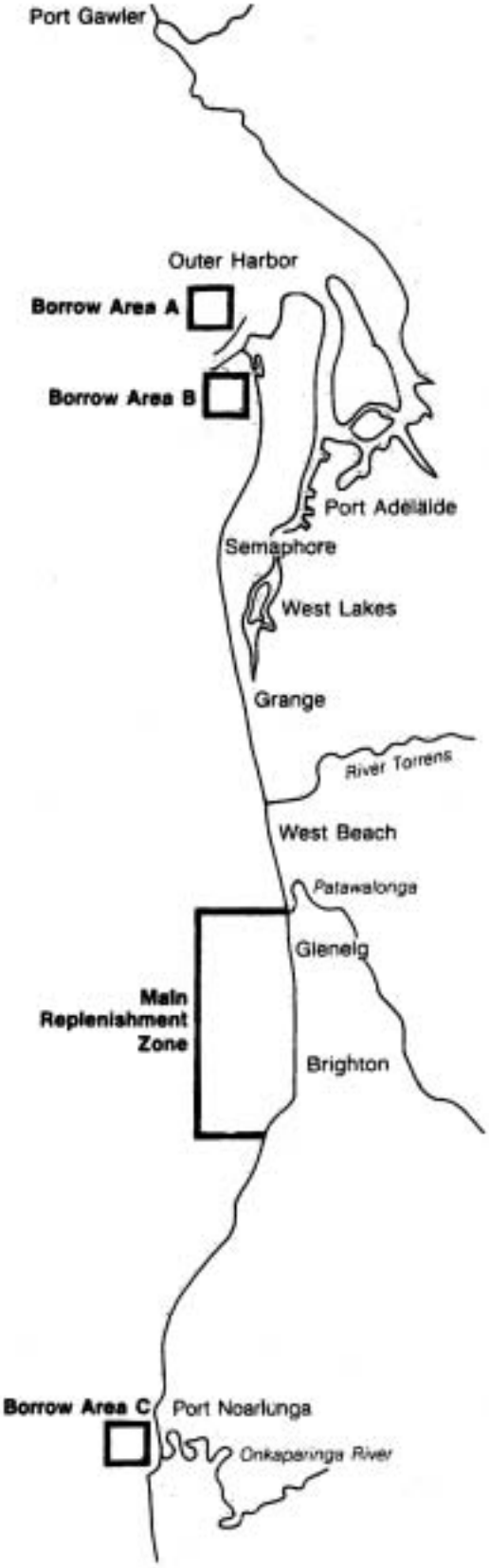
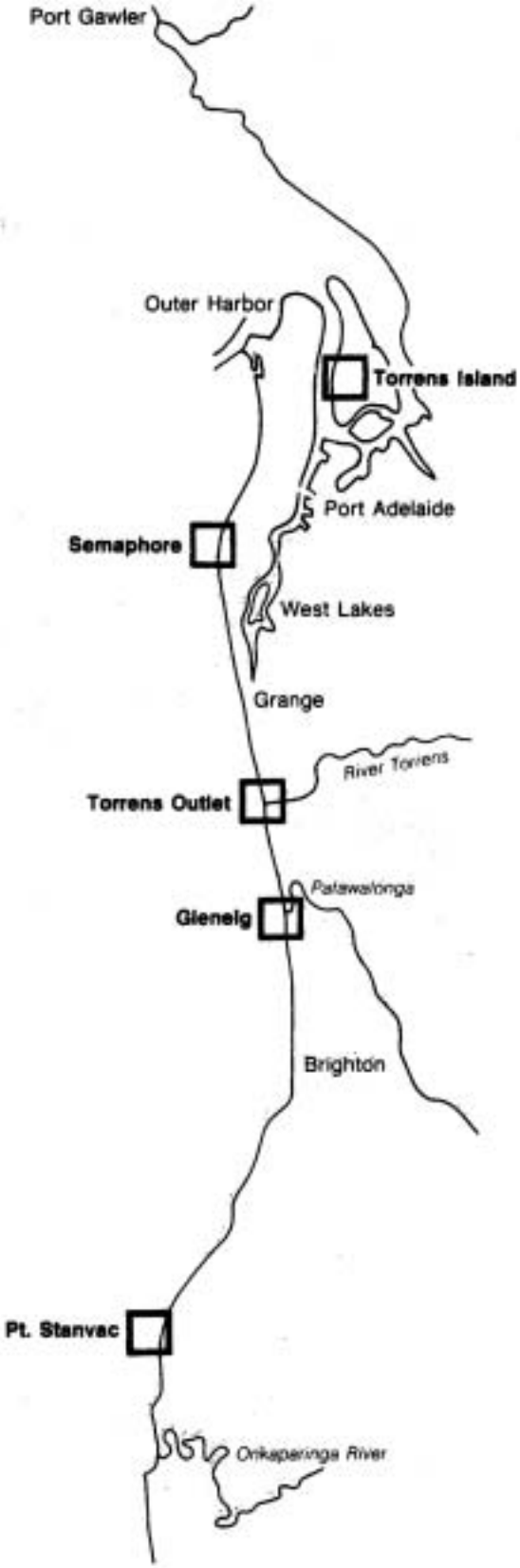


FIGURE 53 ONSHORE SAND SOURCES ASSUMED



The water depths and sediment thickness of the assumed sources are as follows:

Outer Harbor (A)	Partly exposed at low tide to 3 m water depth. The depth of deposit has been tested to 5 m. DMH dredging and seismic information indicate that there is a hard horizon at approximately 6 to 7 m below low water, limiting the depth of material.
North Haven (B)	Water depths are less than 6 m, and the depth of sand is unlikely to be more than 2 to 3 m. Although the North Haven deposit is probably too fine, possible sources to the south, off Semaphore possibly, may have the same criteria though quite probably an even shallower sediment layer.
Onkaparinga (C)	The depth of the deposit is unknown, assuming that it exists. It has, however, been assumed to extend to depths of 10 to 15 m.

These depths influence the choice of dredging plant. Because of the shallowness of the Outer Harbor deposit, and the limiting limestone layer approximately 6 m below the low tide water level, a large trailer-suction dredge would be unsuitable. A smaller, shallow-draft vessel could be used, but the material is variable and this type of dredge is not suited to selective working. Either bucket or cutter-suction dredges could be used, but both these types would have to be matched by a large number of barges for economical operation over the distances involved. If the untested area seaward of and adjacent to the Outer Harbor deposit is suitable, it could be worked by a medium-sized trailer-suction dredge.

The supposed deposit at North Haven is too shallow for a large trailer-suction dredge, but could be worked by a bucket dredge, a cutter-suction, or a smaller trailer-suction. It may be somewhat too exposed and too far from refuge for efficient use of a bucket dredge. All the plants would be too large to enter North Haven.

The assumed range of depths of the supposed Onkaparinga deposit permits any of the plants to be used, but the site is more exposed, and could only be worked safely and economically using dredges of the trailer-suction type, preferably the larger ones.

The Department of Marine and Harbours has advised that dredging the Outer Harbor source would be in conflict with its plans to reclaim the area for future port expansion. These plans are not definite, and are for a long time ahead. In the circumstances, and assuming that no better source can be found, it would be necessary to compare use values to the state on a present day value basis. This has not been done, but would seem likely to favour the coast protection use.

Quantity Adjustment for Sand Grain Size

The Branch's earlier testing of the Outer Harbor deposit had shown the material to be slightly finer than the beach sand, and an overfill ratio (see Section 2.2.4) of 1.2 has been calculated. The assumption was made that the North Haven material would be similar, and that the Onkaparinga deposit, if it existed, would contain coarser sand, presumed to be at least as coarse as the Brighton beach sand. The Dredging Study's costs, and those derived therefrom and shown in Table 7, are based on effective replenishment quantities rather than on actual quantities dredged. This has given the Onkaparinga deposit a cost advantage over the other two sites. It must be remembered that this is an assumption that, like the existence of this deposit, has not been confirmed.

5.2.2 Dredging Practicality, Techniques and Costs

The Dredging Study considered all forms of sand transportation, including road, rail and pipeline, though these three methods were only considered for source B (North Haven). The three land transport methods all have the cost disadvantage that the sand needs to be got ashore first using a floating dredging plant, with a pipeline to an onshore rehandling station.

This section considers the two duration options (short-term or over 10 years), the various dredging methods, the options based on these, and the comparative costs. Table 7 is almost self-

explanatory, and discussion will not deal specifically with the numbered options, but will be more general with occasional reference to these. The table should be read as an important part of the text.

Although the Dredging Study was only at a preliminary feasibility level, the table can be taken as giving quite reliable costs. Several are based on costing done by dredging contractors, and the consultant was able to check most against actual similar projects.

Environmental factors are not included here. They are covered in Section 5.9.

Short-Term Dredging, or Over 10 Years – Quantity Adjustment

The Branch's original estimate of sand volume for a major replenishment was 2.3 million m^3, and it was estimated that this would be sufficient for 10 years, allowing for the losses due to sea level rise and alongshore transport. It was realised that an operation spread over 10 years would have cash flow advantages, and might also prove less costly in terms of present day values, and the dredging consultant was briefed to include this option, allowing for 3 million m^3 instead of the 2.3 million m^3. The additional volume was to provide for losses during the 10-year period. These losses could be expected to be higher than for the major replenishment because of continued interaction of the sea with the seawalls during part of the period. The Dredging Study thus compared the options on the basis of 2.3 million m^3 for the short-term operation and 3 million m^3 for the 10-year one.

The consultants for the Alternatives Study did a more detailed analysis of the volume requirement (see Section 5.7.4), and concluded that 3 million m^3 was needed for the short-term operation. It was considered that a minimum of 2.7 million m^3 was needed without taking account of losses, and that the additional 0.3 million m^3 was necessary at the start to prevent the dunes being lost before the 10-year top-up. They thus concluded that the same quantity of 3 million m^3 was needed for both the short-term and 10-year options. The figures in Table 7 have been adjusted to allow for the increased volume. The original figures are shown in brackets. These volumes are volumes in place on the beach. Actual dredged volumes may be greater, depending on the overfill ratios.

In the Alternatives Study, a further 'top-up in advance' of 400,000 m^3 for future years 10 to 20 was included for options involving a major replenishment. This was costed by assuming round-the-clock operation of the dredge during its 10th year of operation. Although slightly more expensive than using trucks at the appropriate time, the dredged top-up was preferred for environmental reasons. This review assumes use of the Outer Harbor sand source rather than the unproved alternative ones. The present day values of doing the year 10–20 top-up by dredging would be considerably higher than by using trucks because the dredging costs are higher and because the money would need to be spent earlier. Costing for the Outer Harbor source has therefore been on the basis of a dredged quantity of 3.0 million m^3 (in place) and use of trucks for top-up replenishment between years 10 and 20.

Road and Rail options

For practical reasons these options were only available for the 10-year operation, and were costed only for area B, the supposed North Haven deposit. They would be possible, but more expensive, for the Outer Harbor deposit. The pipeline to shore from the cutter-suction dredge would need to be longer, perhaps with a booster station, and would need to cross under the Outer Harbor shipping channel. For the Onkaparinga source, the terrain, distances and other factors make these methods either impractical or more expensive.

The trucking option would be similar to the present program, with the trucks being loaded from the dredge stockpile at Taperoo–North Haven.

In the rail option, the sand would be loaded onto rail cars at a new shunting siding at Outer Harbor, using a large front-end loader. The sand would be unloaded at Brighton using a conveyor system installed under the track. This would transport it to a storage tank, where it would be mixed with seawater before being pumped to the beach.

At costs of $17.1 million and $27.0 million for the road and rail options respectively, these are the most costly replenishment methods. They have consequently been excluded from further consideration.

Pipeline Options

Transport by pipeline was attractive for several reasons. It would reduce the dependence on weather conditions, which would especially hamper the second, pumping ashore, stage of the sea transport options.

It would provide convenient and economical flexibility of discharge point, enabling the beaches to be replenished evenly, and a sand supply could be quickly directed to a special area. And, if the economics allowed, the pipeline and pumping plant could be retained for future regular top-up replenishment, thus avoiding the need to use trucks.

Both short- and long-term pipeline options were based on an actual surveyed route following the coast and esplanade roads. The consultants considered an underwater, offshore route to be impractical, mainly for maintenance reasons.

Both options were based on long-distance slurry transport technology, using a high (50%) solids ratio, high-pressure positive displacement pumps, and a pipeline with a very high pressure rating at the pump end tailing off to a low pressure rating at the discharge end. Booster pumps en route are not needed for this method. This differs from conventional dredge pumping technology more commonly used in beach replenishment schemes, usually over shorter distances. These operate on a lower solids ratio (15% to 20%) and use centrifugal pumps, which operate at much lower pressures. Several booster stations are usually required.

The high solids ratio used in the method examined means that less energy is wasted pumping water. The consultants considered that the greater control offered by this method reduces the chance of blockages, which are very serious and costly in long-distance pumping. These advantages are partly offset by the extra requirements at the pump end. The slurry would need to be kept well mixed at a consistent solids ratio, using a large centrifuge or stirring tank, which would also take the discharge from the offshore dredge, and would have to be large enough to serve as a stockpile during bad weather when the dredge might be inoperable. The mixture would then need to be put through a trial pipe loop to be checked before it entered the main pipeline. The pumping arrangement suggested consists of a pair of positive displacement pumps working in tandem. Small, low-pressure centrifugal pumps would be included to feed the displacement pumps. It was assumed that this installation would be located seaward of the esplanade road just south of North Haven.

3 years was the minimum practical period for a short-term operation. A 350 mm diameter pipeline would be used, and the economics dictate that it be laid above ground and that it be recovered and sold after the project. As shown in Table 7, the duration increases to 3.5 years for an ‘in place’ replenishment quantity of 3 million m^3.

For the 10-year option a 200 mm pipeline would be used, and would be buried for much of its length. Although the consultant did not address the question of retaining the plant and pipeline for future use, the overheads would almost certainly be excessive for the small annual top-up sand quantities.

During the course of the Dredging Study, the consultant received costed proposals from an Australian and an overseas firm, both using conventional centrifugal pumping systems with booster stations. The costs were very similar to those estimated by the consultants for their preferred method using positive displacement pumps.

The pipeline method is clearly not competitive for the 10-year option, being up to $5 million more than using a custom-built dredge. However, the method compares more favourably over the shorter time period, being within $1 million to $3 million of the cheaper dredging options. The Dredging Study did not cost this method for the Outer Harbor source, and this has since been done by the Branch using the figures provided in the consultant report. This is method ‘8A’ in Table 7.

It was assumed for SA that the pumping station would be located on reclaimed land north of the Outer Harbor channel and that the high pressure pipe would be laid under the shipping channel. An extra capital cost of $0.7 million was estimated, assuming that the underwater length of pipe cost twice as much per unit length as on land. Operating costs were estimated to increase by 11%, because of this estimated increase in the pumping head (due to the additional length) and the consequently higher energy requirement.

The estimated present day value cost of $17.9 million is $1.7 million less than that using a bucket dredge and barges, though the work could not be done economically in less than about 3.5 years. It has operational advantages: it reduces risks considerably, and it would avoid the environmental risk associated with dumping and rehandling sand off the beaches to be replenished.

Types of Dredge

The type of dredge and its availability has a strong influence on the practicality and cost of working the offshore deposits considered here. There are three main types that could be employed for the assumed deposits, either on their own or in separate parts of the operation. These are trailer-suction hopper dredges, bucket dredges and cutter-suction dredges.

Trailer-suction hopper dredges are large seagoing vessels that are most suited to working large, uniform deposits in sea conditions too difficult for other types of dredge. Although they are usually most economical in their larger sizes, a small one with pump-out capability would be suitable for beach replenishment over a 10-year period.

The method is that one or two suction heads, with high-pressure jets to loosen the sand, are trailed on articulated ladders as the vessel sails to and fro across the deposit. A sand–water mixture is sucked into the on-board hopper, where the sand settles and the water overflows. The operation is like that of a giant vacuum cleaner.

The Dredging Study considered a typical large dredge of this type (method 1 in Table 7). This dredge, as with other large trailer-suction dredges, does not have a pump-out capability, because the operating cost is so high that rapid bottom dumping is usually preferred to a time-consuming pump-out operation. The usual method is for these dredges to dump onto a spoil site or into a rehandling basin, from which the material is pumped ashore by cutter-suction dredge. This is the system used in method 2, where the high costs reflect the difficulty of operating a cutter-suction dredge in the exposed waters off Brighton. To avoid this, method 1 includes a moored barge with a booster pump to assist the main dredge, which would be modified for pump-out. The barge would connect to the dredge with a flexible hose, and would itself be connected to a steel pipeline to shore with another flexible hose. Although time-consuming, pump-out is shown to be the cheaper alternative here. It also has the advantage of avoiding sand losses and possible seagrass damage associated with dumping the load and re-dredging it ashore.

All the other trailer-suction options, except method 2, assume direct pump-out from the dredge. The 1600 m Thomas Riley, of method 3, from the Brisbane Port Authority, has this capability, which is proposed for the custom-built 1000 m dredge assumed in all the other trailer-suction options.

The larger trailer-suction dredges cannot work shallow deposits such as at Outer Harbor and North Haven, and could not come as close to shore with their loads. This means that a longer and more costly pipeline would be needed to shore.

Both the consultants and the Department of Marine and Harbours have advised that the contract dredging rates, as shown in the table, reflect current bargain prices, because of an international lull in dredging work. It is uncertain how long this situation will last.

In commenting on the Dredging Study, the Engineer for Deepening suggested that if the State were to have a small trailer-suction dredge built, it would be advantageous to combine a pump-out capability with the ability to dump in shallow water using the ‘split rail’ method. This would enable the dredge to carry out other work economically as well. However, it would cost more to build. This would deserve serious consideration if the method were to be adopted. However, the

additional cost is not one that could reasonably be attributed to a replenishment project, and the cost figures have therefore not been adjusted to take account of this.

The Dredging Engineer also suggested that if the dredge had the ability to dump as well as pump out, this could be used when weather conditions prevented pump-out, and this would avoid costly delay. These loads would then be left to come ashore under natural processes. This is not considered viable, having regard to the vessel's proposed 4 m draft, the dependence on tides, and the results of the dumped red sand experiment (Section 2.5.4).

A bucket dredge with barges for transporting the sand could be used at area 'A', north of Outer Harbor. Despite many disadvantages, this method must be considered because this is the only firmly established deposit, and DMH owns a dredge and two barges that could do the work. A bucket dredge is unsuitable for the other assumed deposits, because it can only work in very sheltered waters, and would be at risk in the open sea away from a refuge. It could be used off North Haven, but the operation would be interrupted frequently by weather conditions.

A bucket dredge uses a 'conveyor belt' of buckets, which, when lowered into the seabed, dig the material and lift it to the surface – rather like a large, floating trench-digging machine. The dredge fills barges that are moored alongside, and which then sail, or are towed, to the dumpsite. DMH has two modern self-propelled 500 m^3 capacity barges, which use a 'split rail' dumping method – the whole barge virtually splitting down the middle lengthways, about a set of pivots. This enables them to dump in water little over 3 m deep.

Despite reservations about use of this sand source, DMH has confirmed that the equipment could be available, depending on other work for it. One difficulty is that the capacity and the speed of the barges is insufficient for economical operation over the distance involved – because the dredge would be operating below its peak efficiency. This could be avoided by purchasing another barge and also upgrading the engines of the present ones to increase their speed. Another, more important, difficulty is that of getting the sand ashore onto the beaches. Method 9 of Table 7 is based on this method, and assumes that the barges dump in pre-dredged rehandling basins off the replenishment beaches. The sand would then be dredged ashore by equipment similar to a cutter-suction dredge, but mounted on a piled platform to make it less dependent on sea conditions.

The Dredging Study did not include a long-term option for use of bucket dredges and dump barges for the Outer Harbor source, and this has since been estimated by the Branch using the figures from the consultants' report. The estimate cannot be reliable because it depends on DMH retaining the plant over the 10-year period, and this is uncertain. It was assumed that the work would be done without adding to or upgrading the existing equipment, and consequently that it would be at reduced efficiency. Establishment costs, though still low, would become more significant – $0.5 million has been allowed for this, spread over the 10 years. In practice, work might only be undertaken every 2 or 3 years, to fit in with other projects, but this would not have much influence on the present day value. The pump-ashore platforms, which form most of the establishment cost, are assumed to stay in position.

The Branch's estimate for the present day value of the cost is $19.5 million. This reduces to $17.4 million if DMH were to acquire a third dump barge and could make this available on a hire basis to increase the efficiency of the operation. However, the former figure would be more likely to apply, and is included in Table 7 for this method (denoted 9A in the table). Since the cost is virtually the same for the quicker operation, the only advantage would be in cash flow and this is less important here, because much of the expenditure would be between Government departments. The quicker option would be preferred for practical reasons and because the protection would be provided sooner. And, as already noted, it could be done at less cost by pipeline.

Method 14, which is also based on the bucket dredge and dump barges, assumes that the sand is dumped as near the shore as possible, and that it will come ashore under natural beach processes. The trial dumping, described in Section 2.5.4, was done to test whether the sand would be likely to come ashore over a practical time period. Results of the test have been encouraging but, having

regard to the dependence on tides, and to possible effects on the seagrasses, this method is no longer considered to merit serious consideration.

While this method could be seen as attractive to keep DMH plant and operators employed (assuming that other work is not available), such factors must remain secondary to obtaining more suitable sand. The Outer Harbor sand must be regarded as only marginally suitable, and this would introduce a risk that should be avoided if at all possible.

Cutter-suction dredges are very common, especially in the smaller sizes. The pumping plant is supported on two pontoons, and a ladder, pivoted between the forward part of these, carries the suction pipe and the mechanical drive for its cutter-head. The position of the cutter-head is controlled by lowering or raising the ladder, by swinging the dredge from side to side against its anchors and by advancing it into the deposit by winching it forward, again using the anchors. The larger dredges may have one or more 'spuds'. These are jacked down onto the seabed and are used as a pivot and for moving the dredge forwards. Some dredges can be jacked up on their spuds to form a platform that is above the water and the influence of waves, but these are specialist craft and would be too costly to bring in for secondary handling of replenishment sand to shore.

Cutter-suction dredges can be used with barges to transport the material, but this is not usual and can lead to excessive water turbidity and environmental problems. It is also only possible with some sediments. These dredges are most suited for transporting the dredged material a kilometre or two by pipeline, or further using booster pumps.

Smaller dredges of this general type often do not have a mechanical cutter, but instead use high-pressure water jets to loosen the material. Other variations are the use of submersible pumps (a specially insulated pump and motor unit that is lowered to the seabed) and 'jet pumps'. The latter are worth a mention here because of their recent application in coastal dredging in the US. The suction is induced by the venturi principle, with a barge or shore-mounted pump pumping water through the venturi where the induced suction draws in a sand–water mixture. The flexible pipe returns to a booster pump on the barge or shore station, from where the mixture is pumped to its destination. The action is assisted by high-pressure jets to stir up the sand at the intake. The advantage is that the intake is connected to the barge or shore platform by a flexible hose, and dredging is consequently not as dependent on sea conditions. Locational control of the venturi poses some problems, though propulsion can be achieved by use of air flotation and directional water jets. These devices would seem to be best where they can operate in craters within fairly thick sand deposits – to reduce the need for mobility and to benefit from a flow of sand toward the unit. Providing that a few practical problems can be resolved, they have very obvious future application for dredging entrances to small boating harbours. Dredging of these must often be done in the surf zone where wave action makes other plants less suitable, and where a jet pump would disrupt boating traffic less than a cutter suction dredge with its several anchor cables. A jet pump has been considered for the Patawalonga. It is possible that the cost of method 11 of Table 7 could be reduced by using jet pump technology to dredge the sand ashore – in place of a cutter-suction dredge.

5.2.3 Dredging Conclusions

The following conclusions can be drawn on the use of an offshore sand for beach replenishment.

1. Dredging from an offshore source is practical and, at \$4.2 to \$5.6/m^3 for options using a trailer-suction hopper dredge, would be of similar cost per m^3 as the present trucking program (approximately \$5.0/m^3 from Semaphore to Brighton). It is lower in cost than obtaining sand from further sources (eg Torrens Island or from commercial quarries). However, an offshore source suitable for a trailer-suction hopper dredge has not yet been found.
2. The Outer Harbor deposit is in water too shallow for a conventional trailer-suction dredge. At an estimated cost of \$17.9 million (present day value), the cost of using this partly proven deposit could be up to \$5.4 million higher than using the other unproven sources. The Outer Harbor

source could be used most economically by dredging with a cutter-suction dredge and transporting the sand by pipeline, though this would take 3.5 years.

Alternatively, the DMH bucket dredge and barges (upgraded and supplemented with one new barge) could be used, but at an extra cost of just under $2 million. There is no advantage in doing a slower, 10 year, operation using this source.

Sand from Outer Harbor is not ideal, and an uncertain amount would be lost from the replenished beaches.

3. From a cost point of view, there is little to choose between doing a short-term replenishment using a large trailer-suction hopper dredge and using a smaller, custom-built dredge of this type to do the work over 10 years, though the latter would offer cash flow advantages. As noted, this plant is only suitable for sources that have yet to be proved.
4. Dredging sand ashore and transporting it by road or rail would cost more than the other methods considered, and offers no advantages to offset the extra cost – rather the reverse.
5. Use of a small pipeline over 10 years is not competitive for the unproven sources, and is also more costly for the Outer Harbor source.

5.3 STRATEGIES ELSEWHERE

Variations of the basic methods discussed in 5.1.1 are used worldwide and in Australia. The purpose of mentioning briefly what is done elsewhere is to show that the local erosion problem is not unique, and that the methods considered are real ones in practical use. However, it must be remembered that special circumstances apply at each place, and solutions are usually not transferable. Because of this, it is not useful to compare the local situation too closely with situations elsewhere. However, the Branch and the consultants for both the Dredging Study and the Alternatives Study have drawn on experience in Australia and overseas, through contact with those involved and through conferences and published articles.

Emphasis is given to Australian examples, and mainly those at better-known places. Space precludes more than a brief mention of each example.

5.3.1 Australian Examples

New South Wales

Very little coast protection work is carried out, apart from dune stabilisation, which is done on a large scale. The preferred policy seems to have been to spend money on comprehensive studies of the processes at problem areas, and to use this information to prevent development from taking place in hazard zones. Groynes have not been used on the open coast except as training walls for navigable river outlets. With the exception of Cronulla Beach, beach replenishment has only been carried out as a secondary benefit of dredging these outlets. Seawalls have only been used on a small scale, mostly within the Sydney area, behind pocket beaches. These beaches tend to retain their sand regardless of the type of seawall, and a variety of types has been used. The tidal range is much less than at Adelaide.

Beach replenishment has been undertaken at Cronulla Beach, where erosion of the town foreshore had been caused by deflation losses into unvegetated dunes at the other, less developed, end of the beach. Earlier seawalls had caused further loss of the beach. Sand was trucked from the dunes, which are also being stabilised with planting. The benefit of the single replenishment has lasted 5 years to date, and is still good.

Beach replenishment with groynes has been recommended for Byron Bay, but has not yet been implemented. This strategy was chosen following a broad review of all possible methods, during which seawalls and offshore breakwaters were considered and rejected as being respectively only

short-term and too costly. Options such as subsidised insurance and removable housing were also considered.

Groynes have been used, with a small amount of replenishment, at Silver Beach, Botany Bay. Channel dredging had caused a local concentration in wave energy, with consequent erosion. The coastal processes and the effect of structures were unusually predictable, and the groynes are reported to have worked satisfactorily.

Queensland

Like New South Wales, Queensland has placed priority on coastal study and the avoidance of development in unsuitable places, but the Gold Coast development pre-dated this.

Beach replenishment has been the main strategy at the Gold Coast, where it was needed to replace beaches that had been lost as a consequence of the northward alongshore transport being interrupted by the training walls at the Tweed River. Estuarine deposits have been used for the original large replenishment of the beach at Surfers Paradise and also for the smaller operations since then. For the main project, a cutter-suction dredge was used, and the sand was pumped to the beach. As with Cronulla Beach, New South Wales, the beach has lasted well. Another significant replenishment is proposed.

Groynes have been built at various places on the Gold Coast, against the advice of the Beach Protection Authority. Dispute continues about these, and it is argued that they have caused more problems than they have solved. The removal of at least a portion of the largest one, at Kirra, is now being considered by the Gold Coast City Council.

Victoria

The Public Works Department has moved from a policy of building seawalls in Port Phillip Bay, to replenishing beaches and to protecting the headlands that hold some of the beaches against erosion. It has also built at least one groyne recently.

Beach replenishment is mainly done with sand from offshore sources, using a trailer-suction hopper dredge, which dumps the sand offshore from the replenishment beach, and a cutter-suction dredge, which pumps it ashore. Some sand is also trucked in, but only in small quantities. Coarse sand from an inland source is used to provide steeper beaches and to make them more resistant to erosion.

A large groyne was built at St Kilda as part of a project to reclaim land for beachside recreation. The groyne serves to hold the position of the beach, which was supplemented to ensure that there was enough sand to bypass the groyne and hence prevent downdrift erosion.

Headland reinforcement is an interesting strategy, which, while not relevant to the metropolitan coast, is applicable to the southern Adelaide area, especially at Witton Bluff. This feature contributes to holding Christies Beach in position, and further erosion of the beach could be expected if the cliff were allowed to continue to erode. The Coast Protection Board and the Noarlunga Council have recently addressed the cliff erosion problem, and have decided to protect the toe of the cliff and also to flatten its slopes to remove the danger of collapse.

An experimental **floating breakwater** was used at Geelong and has now been placed at Port Melbourne.

Western Australia

In recent years, more groynes have been constructed in Western Australia than in other states. These have generally been in situations where downdrift erosion could be tolerated, at least in the short-term. Replenishment of beaches within and downdrift of the groyne fields has increasingly been needed, as the erosional effect of the groynes has become intolerable.

Beach replenishment has, with one minor exception at Esperance, been associated with existing groynes. Channel dredging at Mandura was combined with beach replenishment (transport by

pipeline), but trucking from nearby beaches has been used in most other instances. A scraper was used to transport sand along the beach for one of the replenishment operations.

Groynes have been built at Fremantle, Busselton, Mandura, Sorrento (North Perth) and Quins Rocks (North Perth). As noted, all have been supplemented with beach replenishment, though only to a minor extent at Sorrento. Several of the groynes have a T-shape to reduce scour caused by wave reflection from the groyne. An interesting observation has been that the popularity of certain beaches (especially Sorrento) has increased since the groynes were built – because of the shelter they give from sea breezes, and because of the platforms they provide for fishermen. This is relevant to the discussion (see Section 5.9) as to whether or not groynes would improve or detract from the Adelaide beaches.

A group of small artificial headlands was built at Kiwana, on Cockburn Sound. This coast is not subject to much wave energy and the structures are small ones – rocks pushed out at low tide. They have been effective, and the Branch is not aware of any erosion associated with them.

South Australia

Past and recent methods for Adelaide are discussed elsewhere in this report. There have been few serious erosion problems elsewhere in the state. However, a few recent strategies are worth noting.

Groynes have been built at Beachport to extend a longer-standing field of small groynes, and a small amount of beach replenishment has been done to assist in filling the groynes. A groyne is also being considered as part of the strategy for protecting Town Beach at Robe.

Beach replenishment has been the main strategy tor Town Beach at Robe where erosion is mainly due to a training wall at the harbour channel interrupting the alongshore transport. Sand has been trucked from the accumulation at this training wall and from elsewhere. Replenishment has also been used to improve beaches for recreation at Port Augusta, Port Pirie, Stansbury, Port Broughton and Port Lincoln. The protection aspect has been secondary for these.

Outside the metropolitan area, seawalls have been built or extended only at Witton Bluff (rip-rap), Christies Beach (rip-rap), Port Noarlunga (grouted stone), North Shields (rip-rap), Port Pirie (gabion mattresses), Port MacDonnell (gabions, which have since been stranded by sand accumulated behind the new breakwater), and Port Broughton (low concrete wall).

Relocation was the favoured option at Surfers (Fleurieu) and at Vivonne Bay (Kangaroo Island), where erosion was threatening property that was mostly undeveloped. A road at Sultana Point, Yorke Peninsula, was relocated inland rather than protected.

5.3.2 Overseas Examples

The range and number of examples is so wide that it is difficult to choose without suggesting a bias towards some methods. Because of the range of erosion situations, it is also difficult to make general observations. However, a few such observations are probably more useful than merely providing incomplete lists.

The western United States seaboard provides a useful general example. Almost the whole of this coast is subject to loss of beaches because of rising sea level and due to man-made and natural interruptions of the alongshore transport. Protection policy, which is mainly determined by the US Corps of Engineers, has changed from mainly building seawalls (and a lesser number of groynes and breakwaters) to beach replenishment. For barrier islands, the more recent strategies have been dune building and locational control of migrating inlets. Beach replenishment has often been combined with harbour dredging, and has not always been successful due to an unexpectedly rapid loss of the new beaches – often because the imported material was too fine.

An interesting example is at Oceanside Beach, San Diego, where the loss of the beach was having a serious economic effect on the resort town (Bagley and Whitson 1982). A major replenishment was done using trucks, and it is interesting to note that this exercise, which involved much more intensive truck traffic than the Adelaide program, was apparently carried out with the whole-

hearted support of the town's residents. The strategy was chosen after an in-depth review of a wide range of alterations, very similar in scope to this review.

In several instances property relocation has been put forward as a viable economic alternative to protection; however, the Branch is not aware of this being implemented anywhere where a substantial amount of property was involved.

It is impossible to review international coastal engineering practice in a few paragraphs, and no attempt will be made to do so. The following are merely a few examples of the recent use of methods considered in this review.

Beach Replenishment. Miami, Atlantic City, Oceanside Beach (San Diego), and many others. A very long list could be compiled.

Groynes. Port Ito (Japan), Port Suma (Japan) – both combined with offshore breakwaters. Many small groynes, but few written up in the technical literature.

Offshore Breakwaters. Winthrop Beach (Massachusetts), Channel Islands (Southern California), Santa Monica (California), Durban (South Africa), Pescare (Italy), Tel-Aviv (Israel), Kaike (Japan). Many other recent Japanese examples.

Headland Control. Singapore.

Floating Breakwaters. Friday Harbor (Washington, US). Also several other NW Pacific coast US marinas.

Artificial Seagrasses. Bourneworth (UK) experimental, Isles of Goeree and Texel and at Eijerlaudse Gat (Holland) experimental.

Seawalls. There is no point in listing these. There are many of a variety of types in common use though, as with groynes, there is a tendency away from structures and towards less interactive methods such as beach replenishment.

Artificial sand bypassing has been extensively used to maintain alongshore sediment transport where this has been interrupted by port structures or small boating inlets. This is usually done by pumping sand from the accumulating updrift coast to the eroding downdrift coast. A few examples are Port Hueneme (California), Channel Islands Harbour (California), Lake Worth Inlets (Florida), Mexico Beach (Florida), and Ruddee Inlet (Virginia). An engineer previously with the Branch did a study tour to the USA to investigate bypassing methods and technology. The results of this are documented in a Branch Technical Report (Beare 1981), which evaluates the various methods for possible use at the Patawalonga Outlet.

Australian examples have been discussed separately and are not included here.

5.4 ONSHORE SAND RESERVES

Adequate onshore sand supplies are essential if the present coast protection methods are to be continued. Work done in the Branch since the Alternatives Study indicates that the main sources at Point Malcolm, Semaphore and the Torrens Outlet could be exhausted in 13 to 15 years time, though a recently discovered deposit at Torrens Island could provide for up to a further 10 years.

The onshore sand sources are shown in Figure 53 (located adjacent to Figure 52 in Section 5.2.1).

5.4.1 Source Beaches – Semaphore/Point Malcolm and Torrens Outlet

The Alternatives Study did not question the adequacy of the reserves on the present source beaches, though it did note that there could be a shortage if a 2 mm/year rate of sea level rise occurred. This effect was estimated using the Alternatives Study value of 0.6 m^3/m of beach length/year for a 2 mm/year relative sea level rise (see Section 2.6.3). The deduced volumes are 1500 m^3/year and 500 m^3/year for the Semaphore/Point Malcolm and the Torrens Outlet sources respectively. These are small in relation to the other assumptions, such as the alongshore transport

rate and the width of the active beach zone, and would not have a significant effect on the period for which the sand reserves would suffice.

The recent Branch estimate is based on the assumption that the program would continue at 100,000 m^3/year, with 60,000 m^3 of this being obtained from either Semaphore/Point Malcolm or from south of the Torrens Outlet. The remainder was assumed to be from south of the groyne at Glenelg or from Port Stanvac. The amount of available sand on the source beaches was estimated on the assumption that the present beach and nearshore profile be displaced landward as far as possible to leave only a sufficient dune for protection of the development behind these beaches. This may be a slightly conservative assumption, because of the wide shallow nearshore expanse, which may contribute towards restoration of the beach once the equilibrium is altered. However, it could also be optimistic in that the profile will shift landwards as sand is taken from the beach, but perhaps with a delayed response. This delay might limit the rate at which sand could be taken. This could be circumvented by resting a source beach for a few years, and perhaps by taking sand before or during the winter months when wave action would cause more rapid adjustment.

It was estimated that the 2.4 km length of coast from north of the Semaphore jetty to south of Point Malcolm could provide 300,000 m^3 of sand and that 250,000 m^3 of the 350,000 m^3 deposit south of the Torrens Outlet could be taken. The remaining dunes at the Torrens Outlet would be needed for protection purposes. The 60,000 m^3/year from Semaphore/Point Malcolm would be offset by the alongshore drift into the area (estimated 30,000 m^3/year average), leaving a net loss of 30,000 m^3/year. On this basis, this beach could provide sand for only 10 years.

There is, of course, a northward transport out of Semaphore as well as into it, though the sand accumulation in the area suggests that this is probably small. This could result in the reserve being less than the estimate. 20,000 m^3/year is assumed to be available each year after depletion, because of the alongshore transport into the area.

Although there has been an accumulation at the Torrens Outlet of approximately half of the estimated alongshore drift (see Section 2.6.3), the future aim will be to reduce this by regular cutting through the entrance (this allows the accumulated slug to move northwards) and by bypassing the beach using earth moving equipment. This beach would thus lose the full 60,000 m^3/year, and only suffice for approximately 4 years.

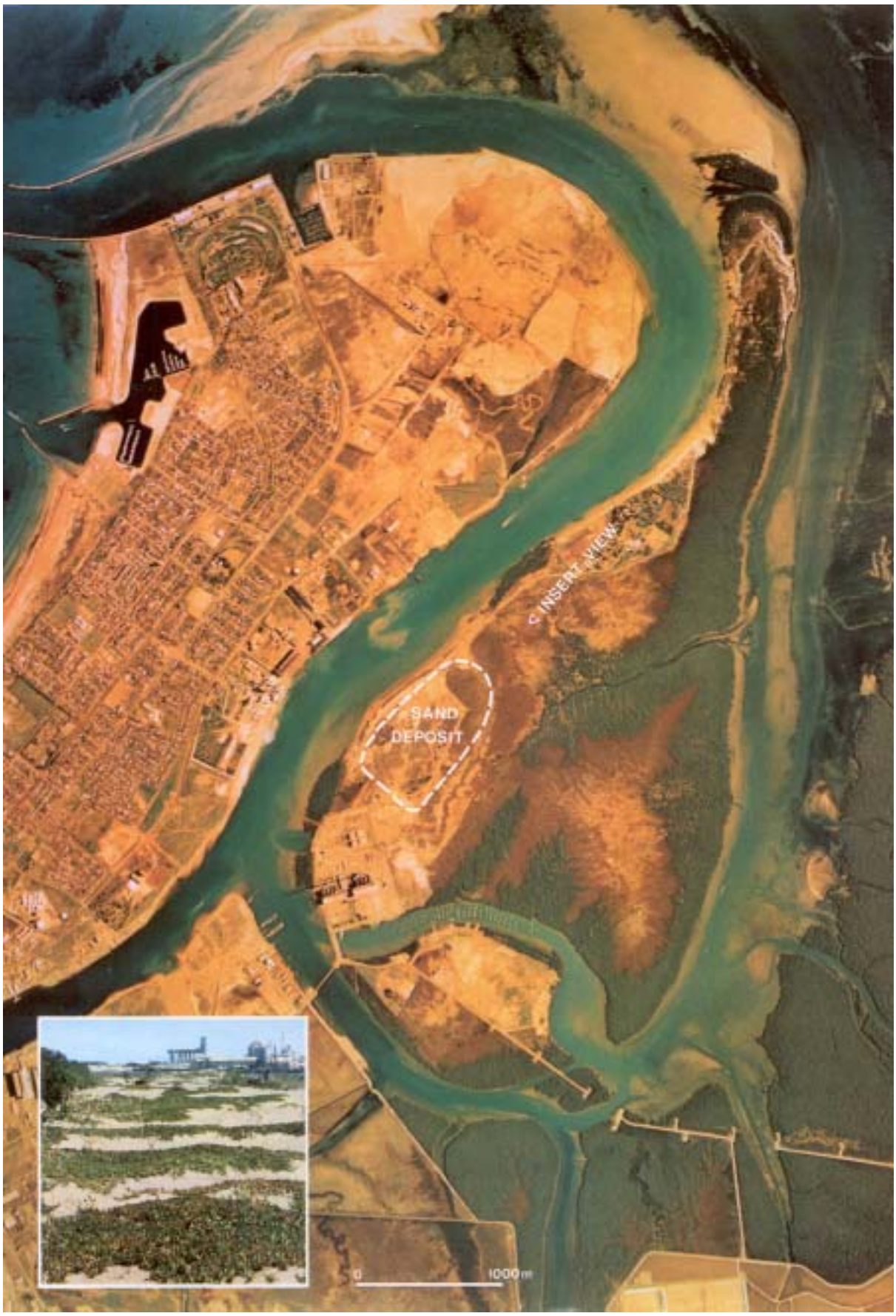
A first impression from experience of taking sand from the northern beaches is that there is much more beach recovery than these figures suggest, and that the reserves could be expected to last much longer. However, the net annual removal of sand has been small. 195,000 m^3 has been removed over the past 5 years, which represents only 45,000 m^3 net loss (9,000 m^3/year average) when alongshore transport into the area is taken into account. This quantity would be too small to notice or measure, being obscured by seasonal and storm beach changes.

5.4.2 Other Onshore Sources

There are two other sources that could be used, though both would involve higher costs, and the environmental impacts of both would need to be carefully considered.

The closest is a large, shallow deposit forming a large part of Torrens Island, and is the property of the Electricity Trust. A small sand mine is already established and this supplies Government and other instrumentalities with sand for pipelaying. This mine is at the limit of its existing lease, and the Department of Mines and Energy is presently considering whether a small additional lease should be given, or whether the sand should be reserved for beach replenishment. A deposit of 500,000 to 600,000 m^3 could be available if other use is not allowed. Initial testing indicates that the sand is suitable for replenishment, but further testing still needs to be done to verify the quantity. The source could provide an annual 60,000 m^3 for 6 to 7 years or possibly longer. Ways of reducing the environmental impact are discussed in Section 5.9.

PHOTOGRAPH TORRENS ISLAND SAND SOURCE



PHOTOGRAPHS WEST BEACH TRUST DUNE REHABILITATION 1977 AND 1979

The other source is a dunal deposit of coarse sand on the shore of Lake Alexandrina. Although 110 km from Brighton beach, this sand may be practical because of its larger grain size (median 0.35 mm). Being coarser than the beach sand, it would form a steeper beach, thus giving better value for money. Unfortunately, the advantage of using coarse sand cannot be quantified and this makes it difficult to justify the considerable extra cost at this stage. The source is also subject to a small mining operation and could be significantly diminished unless arrangements are made to reserve it for future use. The deposit is of approximately 500,000 m^3 and, because of its larger grain size, might be equivalent to at least 10 years supply from the present sources.

The Mount Compass area, south of Adelaide, may be another possible source. This area has not yet been explored, but is known to contain coarse sand.

No other suitable, naturally occurring sands have been located. Failing the availability of the aforementioned two sources (or after they have been exhausted), the replenishment program could only be continued at great expense by using washed sand from commercial quarries, though this would be undesirable because it would affect the supply available to the construction industry, and would probably increase the cost of sand. The present cost of using washed sand from commercial supplies would be approximately \$16/m^3 to \$18/m^3 on the beach. This is much higher than the present costs, which are in the range \$3.00/m^3 to \$5.00/m^3, or than the estimated cost of \$7.00/m^3 for the Torrens Island sand. The cost of trucking sand from Lake Alexandrina would be approximately \$10/m^3 to \$12/m^3.

Surplus sand from the West Beach and Onkaparinga dunes could also be used. However, it is assumed here that environmental considerations would prevail and that, if these sources were to be used, the quantities would be limited to a minimum – perhaps to supplement other supplies and to reduce the overall cost. Both sources have been tested and found to contain suitable sand.

5.5 SPECIAL FACTORS INFLUENCING THE COST COMPARISON

A few special factors influence the cost comparison, some in a direct and obvious way, and others less directly. These need to be explained and the assumptions set out. Costs associated with factors such as beach amenity value and the possible saving on dredging at the Patawalonga and North Haven are set out separately in the cost summary tables later in this chapter and reproduced in the main report summary. Others, such as costs to maintain stormwater outlets, and dune maintenance, are included in the construction and maintenance costs for each alternative.

The factors considered here are: recreational value of beaches, Patawalonga channel maintenance, North Haven channel maintenance, top-up beach replenishment, maintenance of dunes and access for beach-users and boating, and stormwater drainage.

5.5.1 Recreational Value of Beaches

The method of beach valuation and the estimate of a value of loss for the ‘no protection’ option are discussed in Section 3.4. As noted there, the Alternatives Study consultants valued the beaches between Seacliff and Grange Road at \$24,500/100 m for an all-tide beach and \$12,500/100 m for a reduced beach, and obtained corresponding values of \$6,424/100 m and \$3,212/100 m for beaches north of Grange Road. They used these values to deduce present day value of beach loss for the ‘no protection’ option and the seawalls only option, both of which result in beach loss, though at different rates. The present day values for these losses depend on when the losses are assumed to occur, and the difference (\$27.7 million for ‘seawalls only’ vs \$23.3 million for ‘no protection’) is because the ‘seawalls only’ option accelerates beach loss by cutting off the supply that would otherwise be provided by erosion of the dunes at Minda and West Beach.

As discussed in Section 3.4, the method is based on a seemingly reasonable present beach value, and the total present day loss values are probably of the right order – though, as noted in Section 3.4, they may be a little high because of non-linear effects, which the method could not take account of.

The consultants calculated beach gains using the same nominal values per 100 m of beach length. This gave an increase in recreational value of $698,000/year on completion of replenishment, when the entire length of the study coastline would be an 'all tide' beach. For the 'continuation of present measures' alternative the consultants' estimate was that it would take 41 years to achieve full replenishment, leading to an annual increase in beach value of approximately $17,000. It was assumed that the increase in recreational value would apply progressively and equally over the period over which the replenishment took place. The beach gain for the major replenishment options was calculated on the basis of replenishment over 10 years, giving an increase in value of $69,800/year. As discussed in Section 5.7.4, it is now considered preferable to carry out a major replenishment over 2 to 4 years rather than 10 years. The effect of this change on the present day value of beach gain is small, and does not warrant alteration of the consultants' figures.

The concept of additional value for beach gain does not seem to be quite as firm as that for beach loss. Undoubtedly there is added value, but it may not be as high as determined by this method. Another possible approach might be to assume a discretionary power to spend the amount (between $22.2 and $25.3 million, depending on the alternative) on recreational facilities, and to consider the priority additional beach might have in relation to other facilities. On this basis, the beach gain values seem to be high. They are retained for want of better figures, but are not considered to merit the weight that the high value suggests. The present day values of the estimated beach value for the various alternatives are set out in Table 8.

TABLE 8 PRESENT DAY VALUE OF BEACH LOSSES AND GAINS

(Values in millions of dollars, over 50 years)

No.	Alternative	Discount Rate		
		2%	5%	10%
1	No Protection	(50.1)	(23.3)	(8.7)
2	Continuation of Present Measures	11.2	5.1	1.9
3	Seawalls Without Replenishment	(59.3)	(27.7)	(10.4)
4	Major Beach Replenishment	19.8	11.0	5.6
5	Major Replenishment with Groynes	19.8	11.0	5.6
6	Major Replenishment with Breakwaters	19.8	11.0	5.6
7	Hybrid Solution	19.8	11.0	5.6

Data from Kinhill Stearns and Riedel & Byrne (1983). (...) indicates negative values, ie beach loss.

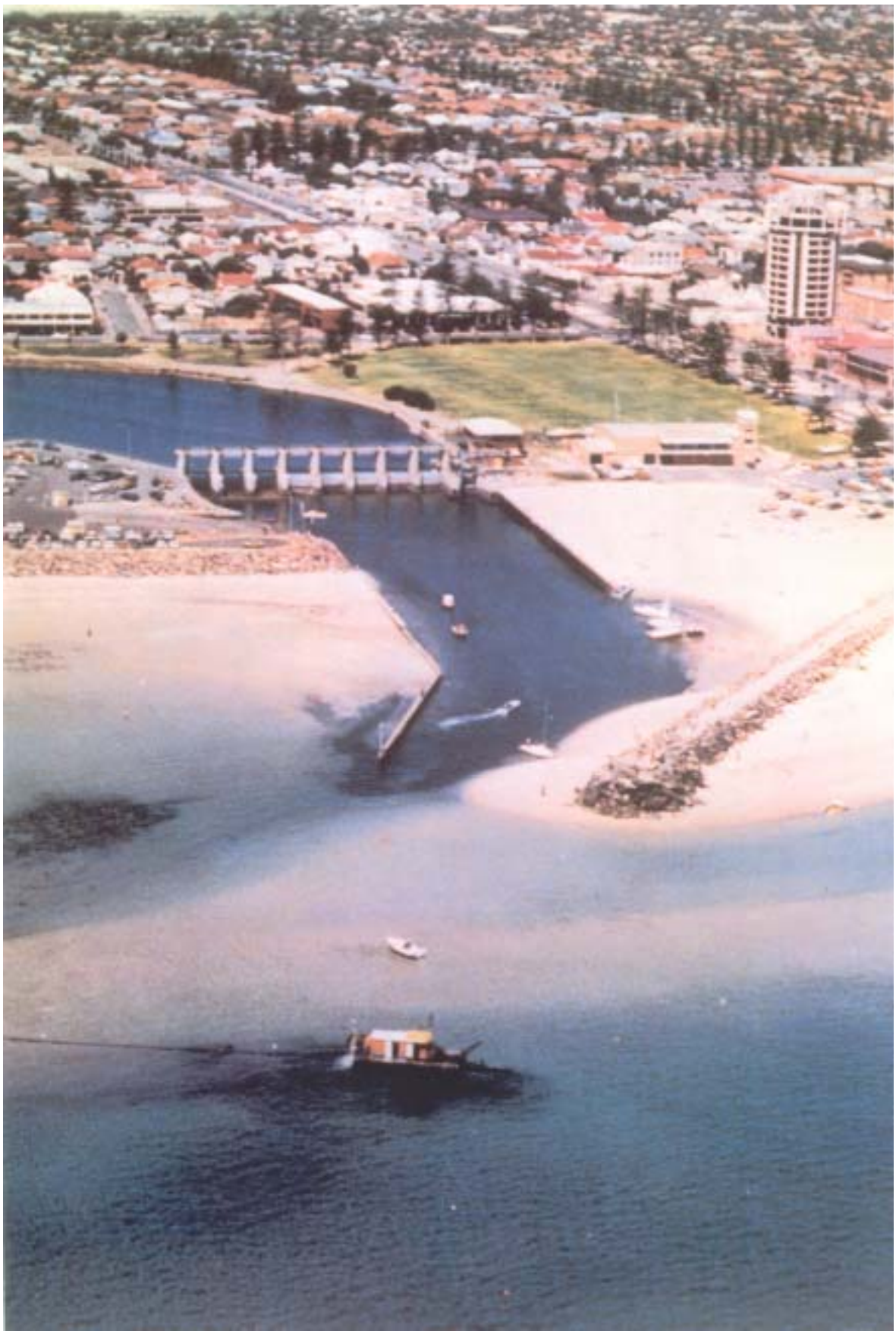
5.5.2 Patawalonga Channel Maintenance

Boating access from the Patawalonga Haven to sea has been an issue since before the groyne was built in 1964. Construction of the groyne only helped for a few years, while the groyne was filling and until sand bypassed it forming an offshore sand bar. Development of a large trailer/boat launching ramp has added to requests for a navigable channel to be provided, as has the increased need for better sea rescue access. The nearby location of the airport makes it especially desirable to have all-tide sea access for rescue vessels.

Dredging is difficult because of the wave conditions, and consequently costly – especially if it has to be weighed against the revenue obtained from the haven and the boat ramp. Nevertheless, a concerted attempt was made in 1979. This failed, mainly because of the difficulty of operating a small plant in the difficult wave conditions and because the incoming sand could not be dredged quickly enough to complete dredging of the underlying clay. However, most of the channel was dug. Although of no immediate benefit, because it has since filled with sand, the channel through the clay remains, and reduces the amount of work that would be required in a future dredging project.

PHOTOGRAPH HENLEY C. 1920 (BEACH USE)

PHOTOGRAPH PATAWALONGA OUTLET DREDGING, 1980



The main lesson learnt from this, and from earlier exploratory dredging work, is that the channel cannot be dredged successfully unless a greater commitment is made and a larger plant used. Maintenance would also be a substantial ongoing cost, and would need to be done either by a larger floating plant than that used previously, or by a permanent shore- or jetty-based installation, perhaps using jet pumps or submersible pumps.

As discussed in the Sediment Budget section (2.6.3), some sand loss is occurring at the Patawalonga despite its natural bypassing. Sand bypassing, as would be achieved during routine channel maintenance dredging, would form a desirable part of whatever coast protection strategy were adopted, and is done indirectly at present by the sand carting program. The initial dredging to create the channel does not, however, have a direct bearing on coast protection, and it is not necessary to consider the cost here.

The assumption has been made that the channel would be dredged and that, if it were, rnaintenance dredging costs and any savings on these need to be considered in the comparison of coast protection alternatives. Methods that reduce the alongshore sand transport would reduce the dredging requirement, and consequently give a saving.

The quantity of sand to be dredged each year approximates the gross average annual alongshore transport, which is estimated to be 80,000 m^3 (refer to Section 2.5.4). For the Alternatives Study, it was assumed that this would require an annual expenditure of $200,000. This is an approximate figure, which could be less if the State were to obtain a suitable vessel for other work. However, the unit rate of $2.50/$m^3$ is typical of commercial rates for this type of work. Dredging establishment costs would be a significant component, causing the costs not to be proportional to the volumes to be dredged. The establishment costs also reduce the cost sensitivity to the volume estimate.

It should be noted that the dredging costs given in the Alternatives Study cost summary tables combine the Patawalonga and North Haven dredging, but that those reproduced here have been altered to delete the North Haven component (see next section). In addition, it has been assumed here that initial dredging of a channel would be done in the year the other strategies commenced, to minimise the possible interference of sand from a major replenishment. Maintenance dredging is assumed to start in year 2. As noted, the cost of the initial dredging is not included here.

It would be more correct to take into account the economic value of the benefits achieved by dredging a channel, since these could be less than the dredging costs. It would, however, have been difficult, if not impossible, to accurately value this benefit, and the work was not considered to be warranted in the present context.

The dredging costs were allocated to the various alternatives on the following basis. Present day value calculations are provided in Appendix A.

Alternative 1 (No Protection)

The consultants assumed that ‘no protection’ meant no expenditure at all on coast protection and consequently that any dredging costs should not be considered as a coast protection sand bypass. No cost was therefore allocated. However, the dredging requirement would be the same as for Alternative 3 (Seawalls without Replenishment), and it could be argued that a similar dredging cost should be allocated.

Alternative 2 (Continuation of Present Policies)

It was assumed that the gross sand quantity entering the channel would remain at 80,000 m^3 a year, and a continued dredging cost of $200,000 a year was allocated.

Alternative 3 (Seawalls without Replenishment)

It was assumed that the present sand movement at the Patawalonga would continue for the first 5 years (with $200,000/year dredging), that it would reduce to half for the next 5 years as the general beach sand supply reduced (ie $100,000 allowed for years 5 to 10), and that thereafter it would reduce to just over a quarter ($60,000 allowed for each year after year 10).

Alternative 4 (Major Replenishment)

The additional sand could slightly retard the alongshore transport (by reducing the effect of wave reflection from seawalls), but this remains hypothetical, and could be offset by having a greater quantity of sand in the Glenelg area. The dredging need is assumed to be the same as for alternative 2 – ie that a continued dredging cost of $200,000 would apply.

Alternatives 5, 6 and 7 (Major Replenishment with Groynes, Offshore Breakwaters or a Combination of These)

The structures would reduce the rate of alongshore transport, possibly to a very small net value. However, there would still be southerly transport into the channel from the North Glenelg beach. The consultants made the assumption that the dredging requirement would be reduced to half as soon as the first structure was built. This structure would be built in the first year and, at the proposed location at The Broadway, would have an almost immediate effect on sand movement at the Patawalonga.

As shown in Table 9, the present day value of the maintenance dredging cost is estimated to vary between $1.8m and $3.6m at a 5% discount rate.

TABLE 9 PATAWALONGA MAINTENANCE DREDGING COSTS

(Present values in millions of dollars, over 50 years)

No.	Alternative	Discount rate				
		0%	2%	5%	7.5%	10%
2	Continuation of Present Policies	9.8	6.2	3.6	2.6	2.0
2A	Maintaining Status Quo					
4,4A	Major Replenishment					
3	Seawalls without Replenishment	3.7	2.6	1.7	1.4	1.1
5	Replenishment with Groynes	4.9	3.1	1.8	1.3	1.0
6	Replenishment with Breakwaters					
7	Hybrid Solution					

Data from Kinhill Stearns and Riedel & Byrne (1983).

5.5.3 North Haven Channel Maintenance

The Alternatives Study took into account the dredging savings that some of the alternatives were considered likely to have at North Haven. However, the argument that structures would reduce the rate of alongshore transport entering this area is no longer considered to be valid within the 50-year period considered. The dredging figures in the cost summary tables presented here have therefore all been altered to remove the costs of dredging at North Haven.

Accumulation has occurred at North Haven (see Section 2.6.3), and the stage has been reached where regular dredging is required to prevent the entrance from silting. The first maintenance dredging, in 1982, showed that the material was too fine to be used for beach replenishment and this, together with a relatively high calcium carbonate content, suggests that at least some of the sand is coming from offshore. An engineering report to the North Haven Trust (Moffat and Nichol 1980) advised that regular maintenance dredging would be needed, at an estimated rate of 80,000 m^3 every 5 years.

The Alternatives Study assumed that half of the sand was from alongshore transport. A cost of $200,000 was allowed every 5 years for all alternatives except Alternative 1 (No Protection) and alternatives 5, 6 and 7 (because of reduced alongshore transport). The exclusion from Alternative 1 is debatable, though no longer relevant. For alternatives 5, 6 and 7, it was assumed that dredging

would cost $200,000 in year 5, would decrease to $150,000 in year 10, then to $100,000 in year 15, and then remain at that level.

The assumption that the structures of alternatives 5, 6 and 7 would have this effect is no longer agreed. The northernmost structure proposed is at West Lakes. This would have virtually no effect on the rates of transport in the Semaphore area, though erosion would occur as sand moved northward and was not replaced from the south. Eventually measures would be needed to prevent this erosion, and these might cause a reduction in the supply to North Haven. However, this would not happen in less than 20 to 30 years, if that early, and would thus have a negligible effect on the present day value of future North Haven dredging.

North Haven dredging is therefore a separate issue, unrelated to protection of the Adelaide coastline, and does not need to be included in the cost comparisons.

Alongshore transport may also have some bearing on silting of the Outer Harbor shipping channel, though this would be small, and would have reduced since North Haven was built. As for North Haven, dredging costs are irrelevant to coast protection in the study area. No allowance was made for them in the Alternatives Study.

5.5.4 Top-up Beach Replenishment

The northward alongshore transport will continue regardless of the protection strategy adopted, although those methods that combine replenishment with groynes or offshore breakwaters would cause it to reduce.

The Alternatives Study included 20 years of top-up in the initial dredging quantities – to replace alongshore and sea level losses. No allowance was made for losses due to natural spreading of the replenishment, which, as discussed in Section 2.6.3, is caused by increased alongshore transport rates during the beach adjustment period. This loss will almost certainly occur, though cannot be quantified. It may mean that more top-up is needed.

It was assumed that the top-up would be trucked from Semaphore after year 20, at a rate of $4.00/m^3 (present day value), that 40,000 m^3 would be needed for the major replenishment (Alternative 4), and that the alternatives of major replenishment with groynes or breakwaters would require a smaller amount of 10,000 m^3/year. As explained in Section 5.2.3, this changes if the Outer Harbor source is to be used. The assumption here, for the Outer Harbor source, is that the trucked, replenishment top-up would commence after year 10, at the same rates as assumed by the consultants.

The top-up costs have only a small effect on the present day value of each alternative. However, it is important to note that trucking would need to be reintroduced for major replenishment options, though at a reduced level if these were combined with structures to reduce alongshore losses.

The costs for top-up replenishment are included in the construction and maintenance costs in the cost comparison tables.

5.5.5 Maintenance of Dunes, Boat Ramps, and Pedestrian Access

The Alternatives Study allowed for ongoing maintenance costs, as distinct from storm damage repairs, which were not allowed for. The costs are relatively small and, although they vary slightly between the alternatives, do not have a significant effect on the cost comparison. It should be noted that maintenance of stormwater outlets was included in the small figure for this general maintenance item, but that this could be a much larger separate item (see next section).

It should also be noted that dune maintenance does not include the initial establishment of the dunes, as would be needed for all the alternatives involving a major replenishment. This was allowed for in the Dredging Study, as a part of the cost of each replenishment method, and has been carried forward into the Alternatives Study. Depending on the replenishment, bulldozing to create the dunes was estimated at between $1.4 million and $2 million, and fencing and planting at $0.4 million. There was a slight present day saving when these were spread over 10 years rather

than done over 1 or 2 years, and this is included in the comparative figures for each replenishment method (Table 7). In practice, this saving might be more than offset by the greater difficulty of establishing a dune, which is, at least in the earlier years, sometimes washed away by storm tides. However, no allowance could be made for this.

The maintenance costs considered here were based on the Branch's experience and estimated at $20,000 a year average. This was applied over the first 10 years to all the options, though a higher figure would be likely to apply for a major replenishment if this were done over 1 or 2 years rather than the 10 years assumed in the Alternatives Study. More work would be anticipated on maintaining both the dune plantings and stormwater outlets.

It was assumed that the cost of maintenance would increase after 10 years for all the replenishment alternatives (mainly because of dune and stormwater outlet maintenance), and that it would remain unchanged for Alternative 3 (Seawalls without Replenishment). The annual cost from the 11th year onwards was assumed to be $50,000 for Alternative 2 (Continuation of Present Policies) and $30,000 for all the major replenishment alternatives. The latter figure is considered to be too low, and has been increased to $50,000. Stormwater drain maintenance would be expected to become more costly with the major replenishment, but to be reduced if this were combined with groynes or breakwaters, because these would be positioned to avoid sand build-up at the main drains.

The 5% present day value for this item is in the range $0.4m to $0.95m. It does not therefore have much effect on the overall cost comparison.

5.5.6 Stormwater Drainage

Stormwater drainage has, at different times before and during this review, been considered either a major item or a minor one. As noted in the previous section, it was treated as a minor item in the Alternatives Study. This would certainly not apply if major modifications had to be made to outlets, and even less so if it became necessary to divert drains to other, less restricted outlets. It has been assumed that neither would be necessary. The outlets would be most affected by the major replenishment option and, in the long-term, by continuation of the present policies, as the beaches built up in front of the outlets. The proposed method would be to allow gaps in the dunes opposite each outlet, and to maintain these gaps by dune fencing and bulldozing as needed. The situation at each outlet would not differ much from that which presently applies at the Torrens Outlet. Some sand build-up would occur, and this could lead to the formation of temporary stagnant pools, which would need to be dealt with.

The major outlets are presently provided with a rock basin and sill, which works well in spreading the flow and avoiding excessive scouring of the beach. Although the basins fill with sand between flows, the pools exist for much of the time, and are unavoidable. The occurrence of these pools and their size might both increase with wider beaches.

The stormwater aspect has not been investigated in depth, and this should be done if it is decided to proceed with a major replenishment. Although it would be worth trying the method assumed here, this may not be successful, and large additional costs could become necessary if major drains had to be re-routed.

In commenting on a draft of this report, the Highways Department considered that the drain levels were such that a reasonable outlet channel should be able to be maintained through a widened beach at only a minor cost. It also considered that it would be undesirable to extend the existing pipes across the new dunes, because of the possibility of damage during storms. The deep sheet piling of the outlets was confirmed, and the Highways Department considered that damage to stormwater discharge structures would only be a small cost for the 'do nothing' option, relative to other property damage.

5.6 SPECIAL PROBLEM AREAS

This review has placed emphasis on the general erosion situation in the southern part of the study area, and on the consequent problem in the Brighton and Glenelg areas. Special problems exist at other places and are influenced by other factors in addition to the shortage of sand and its northward transport. These also received consideration in the Alternatives Study. The information is repeated briefly here, with a few changes. The potential problems associated with stormwater drainage have already been discussed in the previous section, and are not repeated. As previously noted (Section 5.5.3), the silting problem at North Haven is a separate issue, which need not be addressed here.

5.6.1 Minda Home Dunes

The rate of erosion of these dunes has increased as more seawalls have been built at Brighton and as the beaches in front of these have dropped. This is despite the analogous temporary situation discussed in Section 2.6.3 where some recent sand build-up has occurred. The dunes will continue to erode unless the adjacent rock protection is isolated from the sea by a wide replenished beach and sand dunes (though some erosion would be useful, because of the sand this would provide).

The rate of erosion could be expected to reduce according to how rapidly the southern beaches were replenished. For strategies that do not include replenishment, erosion will continue until a small bay is formed between the ends of the rock protection, which would act as headlands preventing further erosion. Although useful and valuable land would be lost, the buildings are located well back from the seafront and would be unlikely to be threatened during the 50-year period considered here – and probably not thereafter.

The situation could be held in the 'seawalls only' alternative, but at the cost of losing this valuable length of beach. Beach loss associated with the 'seawalls only' alternative would greatly increase the value of any remaining small beaches, to the extent that it might be worth allowing erosion if this enabled a beach to be maintained. The beach value might be higher than that of the land, though an equity question would arise because the land is privately owned.

5.6.2 Patawalonga Channel and North Glenelg Beach

The effect of the various alternatives on maintaining a channel at the Patawalonga has been discussed in detail in Section 5.5.2. If a channel is excavated and maintained by dredging sand onto the beach to the north, the cause of the present erosion would be removed. However, additional replenishment would be needed if the beach were to be built up so that the scouring effect of the seawalls was prevented.

If a channel were not dredged, the present natural bypassing would continue, with some loss offshore, as discussed in 2.6.3. The beach loss at North Glenelg will persist because the natural bypassing feeds sand back to the coast much further north, towards the Torrens Outlet. A major replenishment, which would include the North Glenelg beach, would provide a temporary solution. A permanent solution would require sand bypassing, or continued regular replenishment.

Without replenishment, this beach could be expected to deteriorate rapidly, with subsequent failure of the seawall and threat to the sewage treatment plant outfalls. The problem would continue north to the West Beach dunes, which are discussed under the next heading.

5.6.3 West Beach Trust Dunes

The historical erosion of these dunes is discussed in Section 2.6.1, and the 'no action' property threat in Section 3.2.4. The erosion is due partly to the general lack of sand on the beaches, partly to the Patawalonga groyne interrupting the alongshore transport, and partly to the effects of the adjacent rock protection. As with the Minda dunes, a bay would slowly form until limited by the rocks acting as headlands. Similarly, a threat to buildings or structures does exist but is not overriding, and the value of the beach may well exceed the value of the land being lost.

Recent experience suggests that continuation of the present policies would continue to reduce the rate of dune recession, but not entirely prevent it. The erosion could be prevented by a seawall (alternatives 2 or 3), but the associated beach loss would be undesirable because of the large caravan park complex behind. A major replenishment, with or without groynes or breakwaters, would prevent further erosion. An increase in the present replenishment quantities could also possibly enable the situation to be held for at least as long as sand supplies last. However, the trucking is considered to have reached its highest acceptable limit, and additional replenishment here is assumed possible only if matched by a reduction elsewhere, which would not be desirable.

5.6.4 Henley Beach

The Henley Beach area has been a critical one since the Torrens Outlet was cut through in 1936. It has been subject to storm damage – despite its not being at the 'problem' southern end of the study area. It is still vulnerable because of its narrow beaches and because the seawalls, with the exception of the new one between Marlborough Street and Grange Road, are not of a high standard. The swimming pool and the vertical concrete wall at the Henley jetty are at most risk, despite some protection offered by the jetty. This acts as a partial groyne causing a slightly wider beach.

The low beaches are at least partly due to the Torrens Outlet, which has a groyne effect. The effect of the outlet is discussed in Section 2.6.3, where it was estimated that it interrupts approximately half of the alongshore drift. This sand has accumulated south of the outlet and now forms a large source for beach replenishment. Over the past 2 years, the deficit in the Henley Beach area has been partly offset by replenishment between Marlborough Street and Grange Road. Some of this sand would spread southwards, but the benefit would be small over the whole area.

The Torrens Outlet bypasses intermittently, in slugs, depending on the frequency of flows large enough to cut a direct channel across the beach. These cause temporary fluctuations in beach width as they move northwards.

The options considered in this report are not primarily directed to this area, and none offer a certain solution for its problems. The present policy would allow for progressive upgrading of the seawalls and could be extended to include regular bypassing of the Torrens Outlet with earthmoving equipment. It should be noted that use of the southern area as a source of replenishment sand does not effect the natural supply to the north – it merely removes part of the accumulation (see Section 2.6.3).

The protection that replenishment of the southern beaches would provide to this area is uncertain and, even for a major replenishment, could not be expected for several years. It would depend on the rate at which the replenishment would spread northwards, and this cannot be estimated (see Section 2.6.3). It may be necessary to replace seawalls, even if a major replenishment is done to the south, though it would seem sensible to defer this as long as possible pending arrival of the sand from the south. Unless the situation were critical, local replenishment would be better than building seawalls. The consultants considered that a major replenishment to the south would provide adequate protection here, but this may be optimistic.

Alternative 2 (Continuation of Present Policies) allows for replacement of the most vulnerable seawalls in this area. This seawall replacement was not included in the Alternatives Study but is included in the present costing as the work is considered necessary and has already been programmed. Alternative 3 (Seawalls without Replenishment) allows for replacement of all the older seawalls within 6 to 12 years time.

Groynes or breakwaters, if used with replenishment, would enable protection of the main trouble spots near the jetty, but could put added stress on some of the other seawalls. Since the replenishment is not proposed this far north, these groynes would only be built if or when the effect spreads to here, and would only be likely to be built in 15 to 20 years time. The Alternatives Study reported that these alternatives would provide complete protection for the whole Henley Square area, though this assumed proven operation of groynes to the south and an abundance of sand, which was assumed to have spread northwards from the major replenishment. Since the rate of

spreading and the performance of the structures are uncertain, it is difficult to support the argument that complete protection would be provided for this area.

Either continuation of the present policies or a major replenishment, combined if necessary with some new seawall construction and bypassing of the Torrens Outlet, would provide better protection in the short term and gradual improvement in the longer term.

5.6.5 West Lakes South

Some early development from the Grange jetty to the south of the West Lakes development is very close to the seafront, as is some of the earlier West Lakes building. This coast has proved reasonably stable, with storm erosion of the dune face being roughly equal to the seaward dune building achieved by the Woodville Council. However, the margin is narrow at places and could be lost due to sea level rise or possibly in a succession of stormy seasons.

As for Henley Beach, the strategies considered are not primarily directed to this area, which relies mainly on the dunes being kept in good condition and on a flow-on from strategies to the south. The timing and extent of any benefit from replenishment to the south are uncertain, because of the uncertainty as to how rapidly the replenishment would spread. The Alternatives Study comment that a major replenishment further south would provide good protection for this area may be optimistic. However, other protective measures could be applied if the coast were to start receding before the benefit of replenishment was felt. Local replenishment might offer the lowest cost temporary solution, which is all that might be required. Seawalls might need to be considered but could become redundant when the effect of replenishment is felt, and they should obviously be deferred for as long as possible.

The consultants' view that groynes or offshore breakwaters (with replenishment further south) would provide good protection for this area is questioned, because any unpredicted erosion associated with the structures would immediately threaten property – there being no seawalls or buffer to prevent this. Admittedly, the structures would only be built if earlier ones to the south had proved successful and if sufficient sand had moved into this area from the major replenishment. However, it is necessary to consider the full strategy here, and the idea of using groynes or breakwaters without seawall backup at the likely erosion areas is disquieting – especially having regard to the very wide groyne spacing considered.

For the present, continuing the successful sand drift fencing seems to be the best strategy for this area.

5.7 THE COAST PROTECTION ALTERNATIVES

The commissioned Alternatives Study covered all coast protection methods that could be practical for Adelaide, and most of the material here is drawn from that report. Some changes have been made, mainly to take into account information that has become available since the Alternatives Study. These changes are explained under the appropriate heading.

The main reasons for the figures in the cost summary (Table 10) differing from those from the Alternatives Study are:

- The major replenishment has been based on use of the Outer Harbor sand rather than on an unproved source – costs based on the latter are included as alternatives 4A, 5A, 6A and 7A.
- Costs of continuing the present sand carting are higher, because of the assumption that the present beach sources would be exhausted, and that more distant and more costly sand would need to be used in later years.
- The major beach replenishment sand volume has been increased for those alternatives where groynes or offshore breakwaters are used – to take account of the uneven sand distribution and the extra coast length.
- The discounting method has been applied differently (see Section 5.1.3).

TABLE 10 CONSTRUCTION AND MAINTENANCE COSTS OF STRATEGIES

Present day values at 0%, 2%, 5%, 7.5% and 10% discount rates

No.	Alternative	Present Value of Costs, $m						
		Costs Over 50 Yrs.					First 20 Yrs.	
		0%	2%	5%	7.5%	10%	0%	5%
1	No Protection	–	–	–	–	–	–	–
2	Continuation of Present Measures	38.9	23.8	13.4	9.4	7.1	12.5	8.2
2A	Maintaining the Status Quo	23.1	15.0	9.1	6.8	5.4	9.6	6.4
3	Seawalls without Replenishment	17.1	13.8	10.4	8.5	7.1	16.0	10.1
4	Major Beach Replenishment (OH)	28.9	24.8	21.6	20.2	19.2	22.6	20.3
(4A)	(from new offshore source)	(25.8)	(21.9)	(19.1)	(17.9)	(17.1)	(19.5)	(17.8)
5	Major Replenishment with Groynes (OH)	29.3	26.8	24.4	23.1	22.0	27.2	24.0
(5A)	(from new offshore source)	(26.3)	(24.0)	(22.0)	(20.9)	(20.0)	(24.2)	(21.5)
6	Major Replenishment with Offshore Breakwaters (OH)	34.3	31.0	27.6	25.7	24.2	32.2	27.2
(6A)	(replenishment from new offshore source)	(31.3)	(28.2)	(25.1)	(23.5)	(22.2)	(29.2)	(24.7)
7	Hybrid Solution (OH)	33.9	30.4	27.0	25.0	23.5	31.8	26.5
(7A)	(replenishment from new offshore source)	(30.9)	(27.6)	(24.5)	(22.8)	(21.5)	(28.8)	(24.1)

'OH' indicates use of the partly proven Outer Harbor sand source.

'New offshore source' – a suitable source yet to be found.

To avoid confusion, discussion of costs is based solely on a 5% discount rate (see Section 5.1.3), except where the effect of the discount rate is discussed under the special heading in 5.10.2.

The sub-section heading numbers (eg 5.7.3) and the alternative method number (eg Alternative 3, Seawalls without Replenishment), are chosen to be consistent with the numbering of methods in the Alternatives Study.

For convenience in use of this report, all significant factors relating to each alternative are included under each alternative sub-heading. Summarised information from the next chapter on environmental and social effects is brought forward for this purpose. This and the effects on special 'problem areas' are discussed under the 'advantages' and 'disadvantages' headings.

Alternatives 2, 4 and 5 (Continuation of Present Policies, Major Replenishment, and Major Replenishment with Groynes) are fully described and discussed under their separate headings. The other strategies do not warrant such a full treatment, because they either do not deserve serious consideration for various reasons or, for alternatives 6 and 7 (Major Replenishment with Offshore Breakwaters, and the Hybrid, Groyne/Breakwater Solution), because they are similar to Alternative 5 but more costly. Design criteria and costing information is provided for all alternatives.

This description and discussion of the alternative methods should be read after, and in reference to, Section 5.1.1, which relates three basic coast protection philosophies to the local situation, and to Section 5.2, which discusses the dredging aspects of a major beach replenishment.

Reasons for exclusion of the following strategies or methods are given in 5.1.2:

- 'no action' and relocation (note that the former is still included as Alternative 1 here);

- groynes without beach replenishment;
- artificial seaweed; and
- shaped offshore dredging.

Present day values of the costs of the alternative methods are in Table 10, which is reproduced in Appendix B.

5.7.1 No Protection (Alternative 1)

Although already noted as a strategy that can be dismissed from serious consideration because of the high value of property loss, this option is included here to maintain consistency with the Alternatives Study and its numbering system. Only a brief discussion is warranted. This option was included more to provide a context for the protection strategies, and as a yardstick against which their costs could be measured, than as a serious alternative in its own right.

Methods, Assumptions and Costing

The method used by the consultants was to estimate the rate of beach loss, and consequently the timing of seawall failure and damage to roads, services and property. Both this and the separate valuation of beach amenity (and the present day value of the loss of this for the 'no action' alternative) are set out in 3.1.4 and 3.4 respectively. The approach is necessarily hypothetical, mainly because an absolute of no action must be assumed but would never apply in practice – roads and services would be repaired, and seawall failures would be attended to.

A total property loss of $28 million was estimated for the 50-year period, with a present day value of $10.3 million. The latter figure has been increased to $13 million (as shown in Table 8) to include the value of lost seawalls (see Section 3.3). These were not taken into account in the Alternatives Study valuation of property loss. These values, together with the estimated present day value of beach amenity loss ($23.3 million), show that protection strategies with present day cost values in the range of $10 million to $20 million are justifiable.

Advantages and Disadvantages

The option is clearly unjustifiable, and discussion under this heading is unwarranted, except to note the effects on problem areas and the environmental and social effects.

The Minda and West Beach dunes would recommence their previous rapid erosion soon after replenishment to Brighton and North Glenelg ceased. There would be no sudden effects at Henley Beach or West Lakes South, and the present situation would gradually deteriorate, first at Henley, where seawall damage could be expected between 5 to 20 years, and later at West Lakes South.

The only beneficial effect might be that it could become easier to maintain a dredged boating channel at the Patawalonga, because of the reduced quantity of sand on the beaches in its vicinity. However, the reduction might only be a small one, and might not occur for several years.

Table 13 shows no adverse environmental or social effects for this alternative, though only because the parameters used are not relevant in this case, except the beach amenity loss. There would be extensive housing relocation, disruption of access and services, degraded, rock-strewn beaches and, to be realistic, considerable construction activity in reinstating the seawalls and repairing roads and services.

5.7.2 Continuation of Present Measures (Alternative 2)

This alternative is to continue the present beach replenishment and seawall construction program at the present level of implementation. Allowance is made for the present beach sand sources becoming exhausted and other, more expensive, sand having to be used in the more distant future. A lesser variation of this alternative – to move just enough sand to maintain the present level of protection – is also considered.

Figure 54 shows this alternative at year 50.

FIGURE 54 ALTERNATIVE 2, CONTINUATION OF PRESENT MEASURES (KINHILL STEARNS AND RIEDEL & BYRNE 1983)

Methods, Assumptions and Costing

The method is to continue carting sand by truck to the Brighton and North Glenelg beaches, and eventually to include those in between. The aim would be to build up progressively beaches and dunes in front of the seawalls. At the rates assumed, a result similar to a major beach replenishment (Alternative 4) could be achieved in 40 to 50 years depending on the program and the accuracy of the sand loss estimates. The consultants deduced that it would take 42 years, but 50 years is more likely for the program as assumed here. Either must be considered as only approximate estimates. As for all alternatives, the sand loss has been assumed to be 40,000 m^3/year (30,000 m^3/year alongshore transport northwards out of the area, and 10,000 m^3/year due to sea level rise – 5,000 m^3/year for Brighton to West Beach and 5,000 m^3/year for West Beach to Semaphore Park).

Sand would first be obtained from the present beach sources, mainly Point Malcolm/Semaphore and south of the Torrens Outlet, until these became exhausted in an estimated 15 years time (refer to Section 5.2.1). Sand from south of the Glenelg groyne and from Port Stanvac would continue to be used, and the small, 15,000 to 20,000 m^3 annual replenishment to Henley Beach would continue. The program would be extended slightly to include regular cutting through of the Torrens Outlet, and also bypassing this outlet annually by moving approximately 15,000 m^3 along the beach past it (refer to sections 2.6.3 and 5.6.4).

The annual quantity has been assumed to continue at 100,000 m^3/year of compacted sand (approximately 130,000 m^3/year loaded in trucks), with 60,000 m^3/year of this being obtained from Port Malcolm/Semaphore or the Torrens Outlet, and later from other sources. The other 40,000 m^3/year was assumed to be obtained from Glenelg and Port Stanvac for the first 15 years, with 10,000 m^3/year only from Port Stanvac thereafter. 100,000 m^3/year was chosen because it appears to be the highest level that coastal residents will tolerate, and also because it is just sufficient to provide, in the 50-year design period, comparable protection to the major replenishment alternatives.

The main difference from Alternative 2 of the Alternatives Study is that more costly sand sources are now assumed for later years. The consultants allowed for the present sources being depleted (though did not estimate when), and allowed a slight extra cost for using sand from further north, towards North Haven. This sand has since been found to be too fine and other, more distant, sources are now allowed for in the costing. The assumptions made are as follows:

- Beach replenishment

  Years 1–15 — Use of present sources at an average rate of \$5/m^3 for 60,000 m^3, and at a rate of \$3.50/m^3 for 40,000 m^3.

  Years 16–25 — Use of Point Malcolm/Semaphore at \$5/m^3 for 20,000 m^3, Port Stanvac for 10,000 m^3 at \$3.50/m^3, and Torrens Island sand (refer to Section 5.4.2) at a rate of \$7/m^3 for the remaining 70,000 m^3. The \$7/m^3 allows for access and rehabilitation costs.

  Years 26–50 — Use of Port Stanvac for 10,000 m^3 at \$3.50/m^3, Point Malcolm/Semaphore for 20,000 m^3 at \$5/m^3, and \$10/m^3 for all other sand (70,000 m^3). Sand could be obtained from the West Beach or Onkaparinga dunes for up to \$5/m^3, from Lake Alexandrina (coarse sand) for approximately \$11/m^3, or from commercial sandpits at rates up to \$18/m^3 washed and delivered to the beach.

  The \$10 figure assumes only a small quantity from local sand dunes, and that preservation of these will mainly take priority.
- The Torrens Outlet would be cut through and 15,000 m^3 bypassed mechanically at an annual cost of \$20,000.
- Seawall construction is assumed to cost an annual \$150,000 for the first 10 years, and not to be required thereafter. The Alternatives Study allowed \$100,000/year, but did not include the new seawall at Henley Beach, which has already been programmed.

- $10,000/year is allowed for establishment of the new sand dunes.
- As in the Alternatives Study, routine maintenance work such as repairs to dunes, fencing, walkways, etc, is assumed to cost $20,000/year for the first 10 years, and $50,000/year thereafter as dunes are built up and need more maintenance.

The cost calculations using these figures are provided in Appendix A. The present day value of the cost, at a 5% discount rate, is $13.4 million.

A 'status quo' option. Since only 35,000 m^3/year is theoretically lost from the southern beaches, it would seem to be worth considering a strategy that accepts and maintains the present level of protection, by replenishing this amount to Brighton only. However, problems would immediately arise at North Glenelg and Henley Beach if these beaches were no longer replenished regularly. The concept of a 'minimum' policy to maintain the status quo is nevertheless worth considering further and costing, though there are disadvantages.

The following assumptions are made for this purpose:

- The total annual replenishment would be $80,000 m^3 (40,000 m^3 to Brighton, 20,000 m^3 to North Glenelg (as a bypass), and 20,000 m^3 to Henley Beach).

Years 1–36	40,000 m^3/year would be obtained at $5/$m^3$ from Point Malcolm/Semaphore and the Torrens Outlet, and 40,000 m^3/year at $3.50/$m^3$ from the Glenelg groyne and Port Stanvac. The northern beach sources would be exhausted at year 36.
Years 37–50	20,000 m^3/year would continue to be available from Point Malcolm/Semaphore at $5/$m^3$ (due to alongshore transport into this area), 20,000 m^3/year would be obtained from Torrens Island at $7/$m^3$, and 40,000 m^3/year would continue to be obtained from Glenelg and Port Stanvac at $3.50/$m^3$.

- The Torrens Outlet would be cut through and bypassed annually at $20,000 a year, as for the main alternative.
- Seawall construction would continue at an average annual cost of $150,000 for the first 10 years as for the main alternative, but it would also need to continue beyond this period. The need is difficult to estimate, but will certainly arise as seawalls need to be replaced and as new ones are needed, perhaps for such locations as Minda and West Beach dunes. $50,000 a year is assumed for the remainder of the 50-year period.
- Annual maintenance is assumed at $20,000/year for the whole period.

The cost assumptions are set out and the costs calculated in Appendix A. The present day value of the cost, at a 5% discount rate, is $9.1 million.

The full alternative ($13.4m PDV) is thus considerably more costly than maintaining the status quo ($9.1m PDV) over the 50-year period. However, present day values of expenditures over the first 20 years are much closer at $8.2m and $6.4m respectively. The extra sand in the earlier years is partly offset by a saving on seawalls in the longer term. The overall financial benefit, when compared with taking no action and taking beach gains into account, is slightly less for the status quo option than for a full continuation of the present measures. The comparison suggests that the better policy might be to continue at the present rates of replenishment for the initial period, and to review the strategy in the future, when its effectiveness would be better known. This also has the advantage that the present localised problems, which are due to too little sand on the beaches, can be resolved before any reduction is made – insofar as these problems can be resolved by this alternative.

It could be argued that use of the cheaper sand at the higher replenishment rates during the first 20 years forecloses options for making a significant saving by reducing replenishment quantities later. However, this argument is offset by the sand being on the beach where it is most needed and where it could be redistributed, if required, at relatively low costs.

In addition, any reduction within the 50-year period would defeat the aim of the strategy, which is to build up sufficient sand to prevent interaction with the seawalls, and the erosion and maintenance problems that arise from this.

The 'status quo' option and gradations between this and full continuation of the present methods are therefore not carried forward into the main comparison of alternatives, though they need to be borne in mind for future review, and for the opportunities that Alternative 2 provides for this. The following advantages and disadvantages are for a full continuation of the present measures.

Advantages

- At a present day value of $13.4m (5% discount rate), this is the cheapest of the alternatives that would result in the southern beaches being retained.
- No large capital expenditure is needed and the program remains flexible from operational and expenditure viewpoints.
- The strategy provides for a progressive and adjustable improvement from the present protection level, which has so far proved to be effective despite a few continuing problems.
- The failure risk in relation to incremental expenditure is least for this alternative – as compared to a major replenishment where sand losses, spreading, and stormwater discharge are not fully predictable, or to using groynes or breakwaters, the effects of which are even more uncertain.
- There would be a steady and gradual improvement of beach amenity.
- The program can be easily varied to include local replenishment or additional seawalls at the more northerly problem areas (Henley Beach and West Lakes South) if necessary.

Disadvantages

- Trucking would need to continue indefinitely at a high level, as would the disturbance and nuisance at source and replenishment beaches. Impacts of seawall construction and trucking associated with materials transport for this would also continue for at least the next 10 years.
- Some present problems, such as erosion at West Beach and loss of beach at Somerton, may continue for several years, though at a reducing rate (note that these will depend also on climatic factors, and that improvements would be in steps, with setbacks, rather than gradual).
- The effect on some problem areas further north (Henley Beach and West Lakes South) will not be felt for many years. Nevertheless, against this, the program could be varied to include replenishment or rock protection at these places if this should become necessary.
- Compared to a more rapid major replenishment, there will be an uncertain period with frequent loss of newly established beaches and small dunes, and with a higher maintenance requirement for the latter.
- In the longer term, sand removal from Point Malcolm and Semaphore could affect the quality of these beaches. In addition, the Torrens Island sand would need to be used, with significant environmental alteration. Environmentally important dunes at West Beach and at the Onkaparinga estuary might also need to be used to offset the high cost of alternative supplies. If commercial sand supplies had to be used, this could reduce the availability and increase the cost of this resource to others.

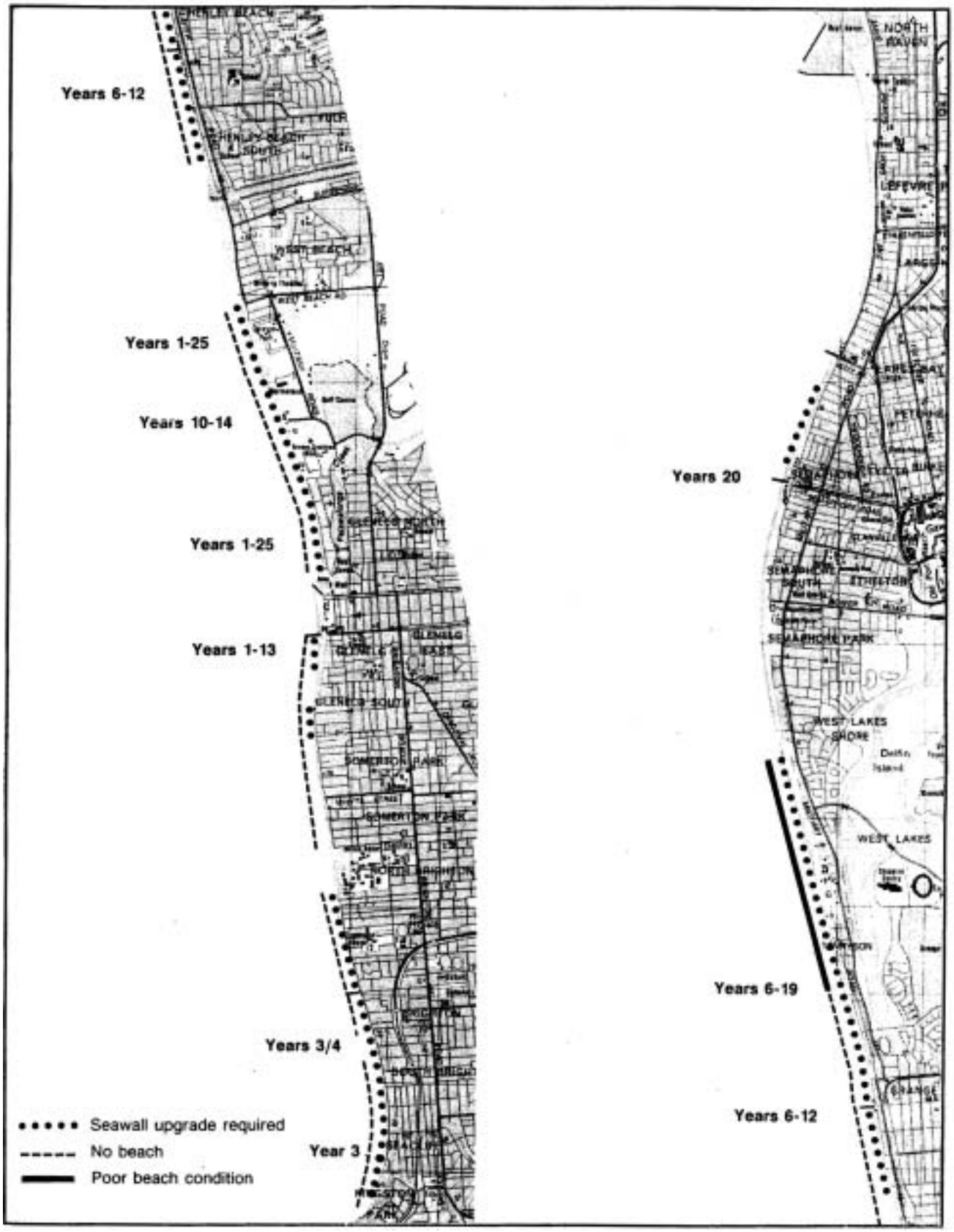
5.7.3 Seawalls without Replenishment (Alternative 3)

This strategy is based on protecting property alone, and only deserves consideration if this is to be seen as the primary aim and if beach loss is to be disregarded. However, this report shows how important and valuable the beach is. This alternative is therefore included for completeness and to provide further context for the cost comparison, rather than as a strategy in its own right.

The strategy would be that seawalls are replaced or upgraded to a standard adequate to withstand storms and the anticipated beach loss associated with this alternative.

Figure 55 shows this alternative at year 50, and also shows when new seawalls might be built or upgrading carried out.

FIGURE 55 ALTERNATIVE 3, SEAWALLS WITHOUT REPLENISHMENT AT YEAR 50 (KINHILL STEARNS AND RIEDEL & BYRNE 1983)



Methods, Assumptions and Costing

The Alternatives Study design and scheduling of future costs were based on the assessment of existing seawalls discussed in Section 3.3 and shown in Figure 48. All seawalls would be brought as close as practical to the standard of those most recently constructed, this work being done over a period of 25 years. Maintenance only was assumed after this, and an annual sum of $140,000 allowed – actually from year 20 onwards.

The consultants affirmed the Coast Protection Board's rip-rap rock seawall design standard (Figure 51B), and confirmed that the method was the most economical and effective in the local situation. Formed concrete units were considered, but were shown to be quite uneconomical in comparison to use of local rock, of which there is an adequate and economical supply.

The possibility of using the Australian invented proprietary design 'seabees' was also considered, because laboratory tests and prototype performance of these (in a lesser wave environment) has shown them to be a very efficient form of protection. The seabees are interlocking units made of either terracotta or concrete, and the finished wall has a 'honeycomb' appearance. Although neat looking, it can be visually unsatisfactory if the pattern is disrupted by poor placement or subsequent movement. Continued use of rock might also give a better overall appearance than introducing sections of a different appearance. Insufficient data was available for the consultants to prepare a comparative method using seabees, but they did suggest that the manufacturer be allowed to put in a bid when a section of seawall is to be built. The units could be prone to plugging by debris, which could make them reflective. This aspect would need to be checked.

The Alternatives Study estimated schedule for seawall construction and upgrading is reproduced in Appendix A and shown in Figure 55. The basis for this is that Class A seawalls are adequate, Class B requires toe-stone and further rock to raise it slightly as well as to compensate for wear and tear, and most of classes C and D would need to be fully replaced with new Class A rip-rap (refer to Section 3.3 for grading of the seawalls).

Costing of new work is based on $1,110/m length, this being a 1982–83 price extrapolated from the Board's local experience. $600/m was allowed for the upgrading of Class B seawalls. As already noted, an annual maintenance sum of $140,000 was allowed from year 20 onwards.

The consultants also included a separate small miscellaneous maintenance item, which has not been altered. $100,000 was allowed at years 5 and 10, and $20,000 a year thereafter.

The only change in the cost calculation is the annual inclusion of cutting through and bypassing the Torrens Outlet for the first 20 years (refer to Section 5.6.4). This was considered worth including in this alternative for the first 15 years, to maintain Henley Beach for as long as possible, but it was not considered worth including for longer, because the benefit would reduce as beaches generally became depleted.

The cost calculations are provided in Appendix A. The present day value of the costs at a 5% discount rate is $10.4m.

The greatest uncertainty in this strategy is the extent of future beach drop, and how soon this would cause failure of the seawalls by undermining. This depends both on the rate of beach drop and on the level of the clay layer that underlies most of the beaches. The rip-rap seawalls are designed to allow for some toe settlement, and the safest are those that have the bottom of their toe within a metre or so of the clay layer. Unfortunately, the height of the clay layer is not fully known. Recent trial coring and excavation (Section 2.2) has shown this layer to be well down at the Brighton beaches, though there is evidence that it is higher at North Glenelg.

Advantages

- This strategy provides the lowest cost for property protection for the 50-year comparison period, subject to the assumptions about the rate of beach drop and the ability of the seawalls to withstand this. Its cost advantage is likely to disappear over a longer period of 50 to 100 years.
- It would provide more positive protection for all the special problem areas than do the other alternatives.

- It would involve less truck movements than any of the other alternatives, and seawall construction would be 'once only' at each place, thus avoiding repeated nuisance to coastal residents.
- Stormwater drainage problems would be minimised, as would dredging maintenance for a boating channel at the Patawalonga.

Disadvantages

- This strategy would involve an even more rapid loss of beaches than if no protective action were taken – because seawalls would be built at West Beach and Minda and would cut off the small supply of sand from these eroding dunes. If the beach loss value is taken into account ($27.7m PDV, from the Alternatives Study), this is by far the most costly solution.
- For most of the coast, this is essentially a short-term solution, which will suffice only until the seawalls are undermined as beaches drop and disappear. There remains uncertainty as to whether or not this would occur within the 50-year design period. If not, it would certainly occur at some time thereafter.

5.7.4 Major Beach Replenishment (Alternative 4)

The aim of this strategy is to replenish the southern beaches with enough sand to establish a beach and dune system that would be able to withstand all but the most severe storms without being cut back to the seawalls; and then to maintain the beach and dunes by regular top-up replenishment.

This strategy recognises that the main problem is due to the dunal supply at Brighton having been cut off and that localised problems are mainly side effects of the seawalls and other structures. The strategy addresses both the main problem and the local effects of structures by replacing the sand supply and by isolating the seawalls, while retaining them as a last line of defence. This alternative is shown in Figure 56.

Methods, Assumptions and Costing

1. The quantity of sand and where it should be placed

This is the first and most important question that must be addressed, but one that cannot be resolved with certainty. Its consideration formed an important part of the Alternatives Study.

The main criteria are the width of the active beach zone (because the sand will spread over this and possibly a little beyond as the beach equilibrium is re-established) and the width of dune that is required to survive a storm of a certain intensity and duration. The consultants surveyed the literature on this rather uncertain matter, and used the method of Swart (1976), checking this against a simpler method from the Shore Protection Manual (CERC 1977).

By comparing these methods with beach slopes from the Branch's beach profile program and with local aerial photography, they deduced that the active beach zone varied between 200 and 600 m, being approximately 300 m where the coast was most stable. They inferred that the beach was accreting where the beach zone was wider, as at Semaphore, and that a tendency to erode was associated with a narrow beach zone. The replenishment volume at the southern beaches had thus to allow for establishment of the wider, stable active beach zone – from the present 100 to 200 m to approximately 300 m.

The sand volume calculations were carried out assuming a replenishment sand similar to that on the beach, and assuming a minimum dune width of 10 m to remain after all but the most severe storms. This is partly to prevent interaction with the rocks and partly to retain a strip of vegetated dune to prevent wind-blown sand loss and the nuisance that this would cause. Swart's method gave a volume of the order of 300 m^3 of sand per metre of beach to withstand a 20-year storm. This was confirmed by the CERC method. This quantity allows for total loss of the dune in a 20-year event, and for a width of approximately 10 m to remain after lesser events. The volume over the assumed 8.5 km length of beach to be replenished is 2,600,000 m^3.

FIGURE 56 ALTERNATIVE 4, MAJOR BEACH REPLENISHMENT (KINHILL STEARNS AND RIEDEL & BYRNE 1983)

The consultants considered that the replenishment sand should be distributed evenly along the two stretches of beach where it was most needed (5,600 m from Kingston Park to the Broadway in Glenelg, and 2,900 m at North Glenelg and West Beach), and that the less vulnerable adjacent areas could be allowed to benefit by natural spreading of the sand. As discussed later, an even distribution may not be the optimum, and the distribution might depend on whether a replenishment were done quickly or over a longer period of 10 years.

Since comparison was made between a quick replenishment and one over 10 years, it was necessary to take into account the top-up volume needed to replace losses due to alongshore transport and sea level rise. These have consistently been assumed to be 40,000 m^3/year to allow for 30,000 m^3/year alongshore transport and 10,000 m^3/year sea level losses. Inclusion of 10 years of this annual quantity in the initial replenishment would enable the 'design beach' to be comparable at 10 years for both these options. It would also give a period without trucking, the avoidance of which forms a strong argument in favour of the major replenishment strategy. The detailed top-up assumptions are discussed later in this section. For all methods, the first 10 years of top-up (400,000 m^3) is included in the main replenishment quantity, bringing this up to 3 million m^3.

The grading and median grain size of the replenishment sand, mainly the latter, can have a large influence on the volume of sand needed. The offshore loss of sand that is too fine is discussed in sections 2.2.4 and 5.2.1, where it was concluded that an overfill ratio of 1.2 (ie 20% extra allowance for losses) should be applied for the partly proven Outer Harbor sand source. For the possible other offshore source (possibly off the Onkaparinga), it was assumed that a sufficiently coarse sand might be found and that an overfill ratio did not need to be applied. The benefit of using a coarser sand is more difficult to define, and perhaps not relevant here, because the possibility of finding such sand seems to be remote – apart from the onshore source at Lake Alexandrina (refer to Section 5.4.2). Nevertheless, the consultants made estimates, based mainly on the steeper beaches that would result. As noted in the Alternatives Study, these estimates are uncertain because of the unknown effect of the coarse sand mixing with the native beach sand. The consultants' conclusions are nevertheless included here to demonstrate the value of coarse sand, if it can be found, and to support the search for such a sand – and perhaps also to ensure that the possible value of using the Lake Alexandrina deposit at some time in the future is not forgotten. Table 11 shows the estimated reduction in alongshore transport rate with grain size. Table 12 shows the effects on initial and top-up replenishment quantities. The figures for coarse sand in Table 12 allowed for a wider dune of 20 m (rather than the 10 m previously assumed) to remain after a storm, as a safety factor to allow for uncertainty in the behaviour of coarse sand.

TABLE 11 ASSUMED VARIATION OF LONGSHORE TRANSPORT WITH GRAIN SIZE

Sand Grain Size (mm)	Longshore Transport (m^3/annum)
0.2	30,000
0.3	22,500
0.4	19,000

From Kinhill Stearns and Riedel & Byrne (1983).

TABLE 12 REPLENISHMENT REQUIREMENTS FOR DIFFERENT GRAIN SIZE

Sand Grain Size (mm)	Initial Replenishment (million m^3)	10-year 'Top-up' (million m^3)
0.2	3.0	0.4
0.3	2.0	0.3
0.4	1.2	0.2

From Kinhill Stearns and Riedel & Byrne (1983).

2. Methods

Since there are no known onshore deposits large enough and because an aim of this strategy is to avoid trucking, an offshore source had to be considered. This is typical of most large beach replenishment projects where dredged material from port deepening is used if available and suitable, or where sand is dredged offshore near the beach and pumped directly onto it, or, failing a suitable nearby source, where sand is dredged at a more distant source and transported to the site by sea or by pipeline. At Adelaide, the port dredgings are unsuitable and too far away, and no nearby offshore source has been found. The last and more costly alternative is therefore the only one available, and there remains some doubt even about this because an ideal source has not yet been found.

Reference should be made to the dredging and transport conclusions in Section 5.2.3. The replenishment strategy considered here is based primarily on use of the partly proven Outer Harbor deposit. However, use of a better source, in a location suitable for use of the more economical dredging method, is also considered and described here as 'Alternative 4A'.

a. If a Suitable New Offshore Source is Found (4A). A reserve of coarser sand in water deep enough for operation of a trailer-suction hopper dredge would provide the most economical way to carry out a major replenishment. The operation, which is described in Section 5.2.2, would be to dredge the material, sail to a mooring off the beach to be replenished, connect to a pipeline to shore and pump the material ashore using the dredge's own pump-out, assisted by a barge-mounted booster pump if necessary. The location of the shore pipeline would be moved several times, and sand distribution on the beach and dune forming would be done using earthmoving equipment.

This would be the most economical method, whether done as a quick intensive operation using a large dredge ($13.6m PDV) or done over 10 years using a smaller, custom-built dredge ($12.5m PDV). The difference in cost is relatively small compared to the assumptions made in deriving the costs. Since there is no strong Department of Marine and Harbours interest in having a dredge of the type for other projects, the choice should be made according to whether a rapid or slower replenishment is best for other reasons, such as to facilitate establishment of the new dunes or to reduce sand losses.

The quick, larger operation provides protection and better beaches sooner, and avoids the difficulties that would be encountered in trying to establish dunes during a prolonged 'intermediate' period. Against this, the rate of sand 'loss' would be higher because the beach alignment is altered more and the beach slope is further out of equilibrium. As discussed in Section 2.6.3, both these factors appear to contribute to an increase in the alongshore transport rate and probably to offshore sand loss. However, a temporary increase in alongshore transport rate is required for the sand to spread, which is needed if the benefit is to extend to the problem areas further north. The increased transport rate is therefore not necessarily a disadvantage, and can be countered by placing more of the sand in the southern part rather than distributing it evenly. However, there is little to provide a guide on this.

The 10-year operation has the advantage that the new problems to be solved, such as maintaining outlets and ramps and building the new dunes, would develop more gradually and there would thus be more time to establish methods. However, as discussed, the problems may be considerably more difficult to solve in an intermediate situation while the sea still reaches the seawalls. The slower option also offers more financial flexibility, though this is partly offset by the initial expenditure of approximately $5 million to have a dredge built.

The 10-year operation has a possible disadvantage in that it could be prematurely terminated because of uninformed public pressure on decision makers. There is an inevitable, apparent sand loss as the imported material is spread over the active beach zone by wave action. Much of the zone is underwater, and the redistribution is interpreted as a loss, with the consequent accusation that, 'Public money is being thrown into the sea.' This is perhaps not so much an argument for or against a quick replenishment as to note an aspect that is usually raised during beach replenishment. Whichever method is used, an intensive public information program would be most desirable.

Another possible factor is the different effect that the timing could have on seagrasses. Although these would be expected to adapt reasonably well to replenishment (because it creates a more natural coastline than the present one), the time for adjustment would be less with a more rapid replenishment, and this could put a higher stress on the seagrasses close to shore. Against this, the scouring associated with the seawalls would be reduced more quickly. There is no experience or information available to give a guide on this.

From discussion with dredging contractors and people with dredging experience, a distinct impression has been gained that it would be preferable to carry out a project of this magnitude rapidly with a large plant, even if economic calculations show a slight advantage in spreading expenditure over a longer period.

On balance, it is considered that a more rapid project would be better for the majority of reasons. However, this is hypothetical at this stage, because a source allowing an economical 10-year project has not yet been found.

b. Use of the Outer Harbor Source. This is the only source that has been tested and found suitable, albeit only marginally so; and only 1 million of the required 3 million m^3 has so far been proved. However, there are large amounts of sand in the general area and it is likely that enough suitable sand could be found. The location of the deposit is shown in Figure 52, and the results of the coring survey in Figure 8. As already noted, the sand is finer than the beach sand, and calculated overfill ratios are 1:2 and higher.

The most economical method would be to dredge it by cutter-suction dredge to a nearby storage-mixing tank, from which it would be pumped to the beaches via an above-ground pipeline. As discussed in Section 5.2.2, it would not be economical to retain the pipeline and pumping plant for later top-up replenishment, which would need to be done by truck as for Alternative 4A. Distribution on the beaches and dune building would be done using earth-moving equipment as for the other offshore source assumed, though would involve slightly less cost because of the pipeline's advantage in distributing the sand. This is taken into account in the costing.

The present cost of the major replenishment using this source has been estimated at $17.9m using a pipeline as described here, and at $19.6m if the work were done using a bucket dredge and barges to transport the sand. The former would be done over 3.5 years and the latter over 2.5 years.

3. Top-up replenishment – methods and timing

As already noted, an average of 40,000 m^3/year has been assumed required, and an amount of 400,000 m^3 has been allowed for in the major replenishment volume to provide for the first 10 years of top-up. The question of later top-up needs to be addressed having regard to three factors:

- It is desirable to avoid trucking for as long as possible.
- Other factors being equal, top-up done in advance is more costly than if done at its proper time – because of the discount rates applied.
- A major replenishment would inevitably result in excess nuisance sand at some places, as occurred at Semaphore before the present sand carting started. It would be undesirable to defer for too long the use of this excess sand for top-up.

If a 10-year project were to be considered, it would be sensible to include top-up for years 11 to 20 in the latter stages of the project, and to avoid trucking for these years, although this would involve an extra present day value difference of approximately $0.3m. Trucking is assumed after year 20. For costing, the trucked top-up is assumed on an annual basis, but this could be varied according to the beach condition or other factors.

For the rapid replenishment, the extra cost of doing top-up in advance would be higher, being approximately $0.4 to $0.5m (present day values) more to do the years 11 to 20 top-up at the conclusion of the main replenishment than by trucking from Point Malcolm/Semaphore or other beaches at the appropriate times.

4. Costing

The annual costs and the calculation of present day values for the range of discount rates are provided in Appendix A. The following cost assumptions have been used.

For Alternative 4 (the Outer Harbor source)

Dredging and beach and dune construction, which includes the first 10 years of top-up: 1st year – $4.75m (mostly installing the pipeline and plant), 2nd year – $8.8m (pipeline, plant and running costs), 3rd year – $4.35m (mostly running costs), and 4th year – $1.26m (running costs less residual value of plant). These figures have been derived from the Dredging Study, the figures there being adjusted to allow for the additional top-up volume and for the extra length of pipe and the extra pumping costs applicable to the Outer Harbor site.

For Alternative 4A (alternative offshore source to be found)

Dredging and beach and dune construction, which includes the first 10 years of top-up: 1st year –$7.8m (establishment and dredging), 2nd year – $6.6m (dredging), and 3rd year – $1.7m (dredging and de-establishment). These figures have been derived from method 1 of the Dredging Study, by extending the time of operation to allow for the first 10 years of top-up.

For alternatives 4 and 4A (common to both)

Top-up replenishment from years 11 to 50 is assumed to be either from Point Malcolm/ Semaphore or the Torrens Outlet at $4/m^3. This is lower than the $5/m^3 used elsewhere for replenishment from Point Malcolm/Semaphore, because of the likely savings due to using the Torrens Outlet and possibly other accumulations nearer to Brighton for some of the volume.

$20,000/year has been allowed for miscellaneous maintenance until year 4 and $50,000/year thereafter (refer to Section 5.5.5). Note that the Alternatives Study used slightly different figures, and that any small difference between alternatives 4 and 4A, due to one being completed before the other, is ignored.

Seawall replacement would still be required at Henley Beach, and the work that is currently programmed is included at $0.35m in each of years 2 and 3. The Alternatives Study allowed for a nominal 2 years of continued seawall construction and spread the cost over the first 4 years at $0.1m a year.

The present day value of the costs for use of the Outer Harbor source (Alternative 4) is $21.6 million (at a 5% discount rate). The equivalent cost for the strategy using a better offshore source (yet to be found) is $19.1 million.

Advantages

- The strategy addresses the source of the problem and removes most of the causes of the present local erosion. As such, it provides the most complete and effective solution.
- For similar reasons, it has the least risk of erosional side effects or un-anticipated environmental impacts. It replaces an original environment, albeit somewhat rapidly and perhaps without sufficient time for normal adjustments to take place. Although the new beach would be built to a 'normal' profile, there would be an initial adjustment to the nearshore zone.
- It is the only alternative that retains beaches and provides a prompt solution to most of the problems at the special areas identified in Section 5.6. Erosion would quickly be halted at all places south of the Torrens Outlet, and the benefit would eventually be felt at Henley Beach and West Lakes South.
- It provides immediate improvement of the beaches for recreational use. This would be especially noticeable in stormy years when the present beach losses would not occur. As discussed in Section 5.5.1, the present day monetary value of the early beach gain is large, being approximately $6 million more than for the slower beach gain of Alternative 2 (Continuing the Present Policies). However, it is debatable as to whether or not this difference should be taken at its face value.

- Notwithstanding the construction and maintenance costs being higher than for continuing the present measures, a major replenishment could provide the highest overall monetary benefit (when compared with the option of taking no action). The overall benefit would be similar to that for continuing the present measures, and would only exceed it if a suitable offshore sand source was available and if the additional value of beach gain was fully taken into account.
- Trucking could be avoided for the next 10 years or perhaps the next 20, depending on how much the avoidance of top-up trucking is considered to be worth. For example, the extra cost of doing top-up replenishment for years 11 to 20 in advance, as part of a major replenishment, would be approximately $400,000 in present day cost value.

Disadvantages

- This strategy is approximately $6m to $8m more costly in present day values (at a 5% discount rate) than continuing the present measures, depending on whether or not a better offshore source is found. This financial disadvantage is very sensitive to discount rate, increasing rapidly with higher discount rates and falling away with lower ones. At a 2% discount rate, the cost is very similar to that of continuing the present measures.
- Large expenditures would be needed in early years.
- Although there is less risk of failure or unpredicted side effects than for most other alternatives, there is a risk of losing some of the replenishment sand offshore, especially if material of marginal suitability, as at Outer Harbor, were used. This risk can be valued by deducing the costs of replacing sand that might be lost if an overfill ratio higher than the assumed 1.2 were to apply. If it were found, say in 5 years time, that an average overfill ratio of 3.5 had applied, then, assuming that the sand deficiency (600,000 m^3 of sand in place) was replaced by trucking from present sources at $5/$m^3$ over the following 5 years, the present day value of the additional cost would be approximately $3.0 million. A further extra cost would also apply because the present sources would be exhausted earlier, and sand that is more expensive would need to be used for later top-up replenishment. The risk of sand loss would decrease if a better sand source could be found.
- This strategy does not prevent the alongshore transport or the consequent need to carry out top-up replenishment by trucking. However, it should be noted that the only strategies that do (replenishment with groynes or breakwaters) have considerable, more critical, disadvantages, and are more costly.
- The extra sand in the southern part of the study area will undoubtedly add to the shoaling problems at the Patawalonga boating channel, and to the added cost of maintaining a channel, were such a channel to be dredged. This disadvantage applies also to continuing the present measures, and is costed in Section 5.5.2 at $1.8m (PDV 5% discount rate), in comparison with use of groynes or breakwaters (alternatives 5 and 6).

This strategy is neither clearly advantageous nor disadvantageous in regard to environmental and social impacts – except for the advantages of improved beaches and reduced trucking. Because of the rapid beach build-up, there is some risk to nearshore seagrasses, but this may well be less than presently applies and not significant in comparison with other losses presently occurring. There would be some beach impairment during the project, but this would be temporary and needs to be considered against the advantage of reducing the trucking impacts. Despite measures that would be taken, the wide beaches and new dunes would make vehicular access to beaches for boat launching more difficult, though this may be offset by added space on the beach for rigging boats.

The pipeline from Outer Harbor (assuming this method) would mostly be above ground and would be an obvious feature of the coast for the 3.5 years of the project. A pipeline below ground would cost more.

Depending on the location of a possible alternative offshore sand source, avoidance of damage to seagrasses or other parts of the marine environment could be a critical issue. Removal of the proven part of the deposit at Outer Harbor would not have significant environmental impacts.

However, a larger area has yet to be investigated and the new area may be more sensitive to disturbance.

5.7.5 Major Beach Replenishment with Groynes (Alternative 5)

The aim of this strategy would be to retain most of the benefits of a major replenishment and, at the same time, to avoid some of the disadvantages. Groynes would reduce the alongshore sand movement, thus reducing the need for trucked top-up replenishment and the costs of maintaining stormwater outlets and a boating channel at the Patawalonga.

The use of groynes without replenishment is not a viable option because there is not enough sand available to 'fill' the groyne field. Groynes without additional sand would result in an uneven distribution of the small amount of sand available, leading to loss of beaches and a worse erosion threat to much of the coast. This is discussed in sections 5.1.1 and 5.1.2.

The consultants for the Alternatives Study considered that groynes with beach replenishment was a suitable means of protecting the Adelaide beach system.

Methods, Assumptions and Costing

The method is to carry out the major replenishment as in Alternative 4, and to simultaneously commence the building of rock groynes. The consultants considered that a minimum of 11 groynes would be needed, and that these could be built over 20 years. The location of the suggested groynes and the construction sequence is shown in Figure 58.

The groyne field proposed is not to conventional groyne design, which involves much closer spacing of the structures (usually between 2 and 4 groyne lengths apart). It was considered that the effect of such close spacing on beach amenity would be unacceptable. Also, the cost of the groynes alone was estimated to be in the range $15m to $20m, and would have made this alternative almost twice the cost of other viable strategies.

The design philosophy was rather to create a number of groyne headlands, which would modify the shape of the coastline by causing it to align more closely with the dominant wave direction. As noted by the consultants, the effectiveness of the protection provided depends on the length and the stability of the 'tail' of the sand wedge held by each groyne. And this, in turn, depends on how close the original alignment was to the wave direction and to variation in this wave direction.

As noted in Section 5.1.1, protection methods that rely on retarding alongshore transport are the most risky, especially at Adelaide, because of the unusually high variation in wave direction. The consultants recognised this and noted that there was uncertainty as to whether or not the groynes, as shown in Figure 58, would work satisfactorily, and recommended that a trial groyne first be built at The Broadway as an experiment to determine the stability of the 'tail' and the length of coast that could be protected.

The consultants' design recognises also that the groynes would result in some parts of the beach between groynes being lost during storms, or if 'normal' conditions should depart from the average for a significant period. These areas, to the north of each groyne, are located, where possible, where there are seawalls considered adequate to withstand the loss of beach. At only one place – Seacliff – did the consultants consider that the seawall would need to be upgraded. This seems to be optimistic, and it is considered that a much greater length of seawall would either need to be built or upgraded. For example, the programmed new seawall south of Marlborough Street would need to be built to provide protection until the replenishment spread that far north and thereafter to provide protection from the erosional effects of the suggested groyne near the Henley jetty. A new seawall would be needed at Henley Beach South, to protect the coast immediately north of groyne number 7. In addition, serious property damage could occur if the vulnerable parts of the coast further north between groynes 8 and 11 were not protected with seawalls. The use of groynes in this area would seem to introduce a much higher risk of erosion than prevails at present, but is difficult to avoid once a decision is made to adopt a groyne strategy.

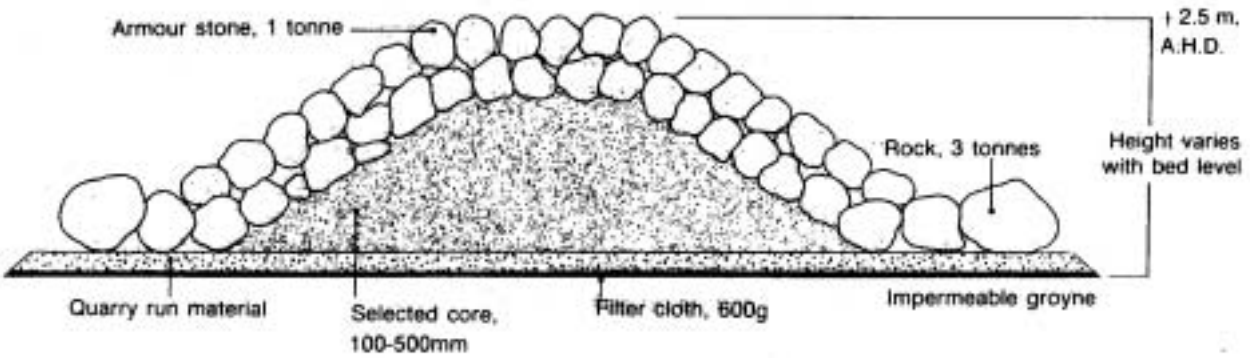
Notwithstanding these comments, the consultants' costing is followed here, with the exceptions of the programmed seawall at Henley Beach and the inclusion of 10 years of bypassing of the Torrens Outlet. Both these items are also included in the costing for the major replenishment, as they are considered to be necessary to provide interim protection of the Henley area, whether major replenishment is combined with groynes or not. Cost sensitivity to the possible need for additional groynes and seawalls is discussed under the 'disadvantages' heading.

1. The groyne design

To be effective, the groynes need to extend beyond the most active beach zone, which the consultants determined to be approximately 200 m. They certainly need to extend beyond the extreme (storm) seaward limit of the offshore bar if they are to interrupt most of the alongshore transport.

The proposal is to use permeable groynes at most places, because they allow some sand through and impose less stress on the coast immediately updrift of the groyne. Permeable groynes are also cheaper to construct. Impermeable groynes are suggested where major drains are to be protected from sand build-up. Figure 58 shows the consultants' suggested location of groynes of the two types. The design of an impermeable groyne, as assumed for the costing, is shown in Figure 57. That for a permeable groyne would be similar, except that it would have large rock throughout and not include a core. A bottom filter is still needed to prevent settlement.

FIGURE 57 GROYNE DESIGN, CROSS-SECTION (KINHILL STEARNS AND RIEDEL & BYRNE 1983)



The proposal to use armour stone of only 1 tonne should be reviewed as this is considerably smaller than the 1 to 3 tonne range used in the seawalls, despite the larger waves that would be encountered at the ends of the groynes. Maintenance of the groynes would be difficult and costly, and it would therefore be unwise to economise on the size of stone. No adjustment is made to the costing at this stage to take account of using larger stone, though this would be significant.

2. Quantity of replenishment sand

The Alternatives Study assumed that the same initial sand volume would be needed as for the major replenishment without groynes (2.6 million m^3), and that the top-up replenishment could be reduced to 0.3 million m^3/year (instead of 0.4 million m^3) for the first 10 years, and that 0.1 million m^3/year would suffice for years 11 to 20. Top-up to year 20 was included in the initial replenishment, bringing the initial replenishment volume to 3 million m^3, as assumed for the major replenishment alternative. Top-up after year 20 was assumed to be by truck at 10,000 m^3/year, at a cost of $50,000/year.

More sand is needed with groynes than without – to compensate for the uneven distribution caused by the groynes, and also to take account of the slight effective increase in the coast length. The latter effect is small, with the total length of the new beaches being approximately 2% longer than the same length of coast without groynes. This is equivalent to approximately 50,000 m^3 of sand.

FIGURE 58 ALTERNATIVE 5, MAJOR REPLENISHMENT WITH GROYNES (KINHILL STEARNS AND RIEDEL & BYRNE 1983)

Estimating a quantity to allow for the uneven distribution is more difficult. A large increase would be needed if the aim were to maintain the same width of dune at the critical points of minimum beach width, but this is not realistic for the groyne strategy. Some increase is nevertheless needed to compensate for the volume of sand 'locked up' in the groyne fillets and consequently not available for protection elsewhere. A conservative assumption might be that a triangular fillet of 100 m against the groyne by 300 m along the coast by 2 m sand depth would be isolated from the coastal processes for all but the most unusual events. For the suggested 11 groynes, this is approximately 300,000 m^3 of sand. It has therefore been assumed that a total additional quantity of at least 350,000 m^3 would be included in the initial replenishment, and this has been included in the costing.

Although the foregoing estimates are very approximate and somewhat arbitrary, no better method exists for this. The allowance may well be too low.

3. Timing of construction, and costing

- The major replenishment was assumed to be carried out as for Alternative 4, though it should be noted that the costs here are based on use of the Outer Harbor source with transport by pipeline over 3 to 4 years. This differs from the Alternatives Study where an unproven new source was assumed by a less costly dredging method carried out over 10 years. Costs for the extra 350,000 m^3 of sand have been calculated on the assumption that the dredge and pipeline method would be extended by approximately half a year for this quantity. The extra cost of $1.2m would occur in the fourth year.
- The consultants estimated the costs of the groynes variously between $408,000 and $542,000, depending on the location and on whether the groynes were permeable or impermeable. Details are provided in Appendix B of the Alternatives Study. The 20-year groyne construction program assumed by the consultants is accepted here and shown in Appendix A. Six groynes are assumed to be built in the first 10 years, and the remaining 5 in the next 10 years.
- For top-up replenishment, 10,000 m^3/year is assumed at an annual cost of $40,000 from year 20 onwards.
- A seawall construction cost of $0.35m in each of years 2 and 3 has been allowed, as for the major replenishment alternative – for programmed work at Henley Beach that is necessary for interim protection. This is in addition to the allowance in the Alternatives Study for replacement repairs and the seawall upgrading at Seacliff ($100,000 allowed in the first year only).
- Annual bypassing and cutting through of the Torrens Outlet has also been assumed for the first 9 years, as for the major replenishment alternative, to ameliorate the situation at Henley Beach for the period until the benefit of the major replenishment is felt. This was not allowed for in the Alternatives Study.
- The miscellaneous maintenance costing of the Alternatives Study has not been altered, though the figures could be low, especially if any maintenance of the groynes became necessary. $100,000 is allowed in each of years 5 and 10, and $30,000 each year thereafter.
- A North Haven dredging saving allowed for in the Alternatives Study is no longer included (refer to Section 5.5.3).
- The possible Patawalonga dredging saving is costed separately as discussed in Section 5.5.2.

The costs in each year used for the present day value calculations are provided in Appendix A. The total present day value of the costs of this alternative is $24.4m (5% discount rate). This reduces to approximately $22.0m if a better offshore source can be found.

Advantages

- Top-up maintenance would be very much reduced, to the extent that trucking could be avoided for the first 20 years, and would only need to be carried out occasionally thereafter.
- Less maintenance or other works would be needed for most stormwater drains, because groynes would be located, where practical, to give minimum beach widths at the drains.

Disadvantages

- There is a considerable risk of the strategy either not providing the required protection at all places, or of it causing erosion at others. This applies especially to the more northerly groynes, which could result in a worse situation from Henley to West Lakes than presently prevails.
- This alternative is $2.8m more costly (in present day values at 5% discount rate) than the major replenishment without groynes – despite the virtual avoidance of top-up replenishment. It is $1m more costly when a dredging saving at the Patawalonga is taken into account.
- If satisfactory protection is not achieved with the wide groyne spacing proposed, it could become necessary to add additional groynes in between. In this event, it is likely that groyne construction would be brought forward to counter the problems. An extra $3m approximately (present day value, 5%) would apply in the event of all 11 groynes being built in the first 10 years, and an additional 5 during the next 10 years. An alternative to this might be the building and strengthening of seawalls immediately north of each groyne. Approximately 5 km of new seawall might be needed (mainly in the northern areas) and 2 km of seawall at North Glenelg might need to be upgraded. The present day value of the cost of doing this work over the first 20 years would also be approximately $3m. The consequences of failure of this method are thus very high. Removal of groynes might involve less cost, but would still be a large item.
- The general standard of protection may be less than without groynes, because of the uneven distribution on sand and the consequently narrow dunes (or absence of dunes) at many places.
- There is more risk of losing sand offshore from the ends of the groynes, than of offshore loss in the absence of groynes. This is because of the cells created by the groynes and the circulating currents that develop in these cells, and because of wave reflection and turbulence caused by the structures.
- There may be an increased safety hazard to swimmers because of the proximity of deeper water on the downdrift side of the groynes and the currents that the groynes would cause.

Environmental and social aspects are not clearly advantageous or disadvantageous for this alternative as compared to a major replenishment alone, though the assessment is subjective, and strong opinions have already been expressed on this. The consultant considered that the groynes might offer an advantage because of the added beach variety and the wide beaches created. Further, experience in Western Australia (refer to Section 5.3.1) indicates that beaches with groynes are more popular than those without, and this is explained as being due to the wind shelter offered by the structures, and their suitability as fishing platforms. The latter advantage would not apply to the same extent in Adelaide, because of the ample provision of more convenient fishing platforms at the jetties. The Coast Protection Board, the Department of Tourism, and the Department of Recreation and Sport have commented on this, the opinion of all being that the structures would be undesirable in appearance and would detract from tourism and from the suitability of the beach for recreation.

5.7.6 Major Beach Replenishment with Offshore Breakwaters (Alternative 6)

The aim of this strategy is similar to that of replenishment with groynes – that is, to retain the benefits of a major replenishment while at the same time reducing the alongshore transport and consequently the cost and inconvenience of trucked top-up replenishment to replace the 'lost' sand. The main difference from the groyne strategy is that alongshore transport is retarded by interrupting the wave energy rather than by the more direct groyne barriers across the beach and surf zone. The reduction in transport by re-alignment of the coast is similar for both methods, as is the 'headland' concept used in the choice of their spacing.

The usual concept of breakwaters – that they provide protection by sheltering the coast from wave energy – applies only to a minor extent because only short lengths of breakwater are considered. Their effect on wave energy would be slight, being more like that of headlands, where wave refraction results in the headlands attracting a disproportionate share of the wave energy and the bays in between receiving less.

Offshore breakwaters have the advantages that they affect the beach appearance and amenity less than groynes and that their local erosional effects are not as harsh. Their longer-term effects, particularly in causing erosion of the downdrift coastline, are similar.

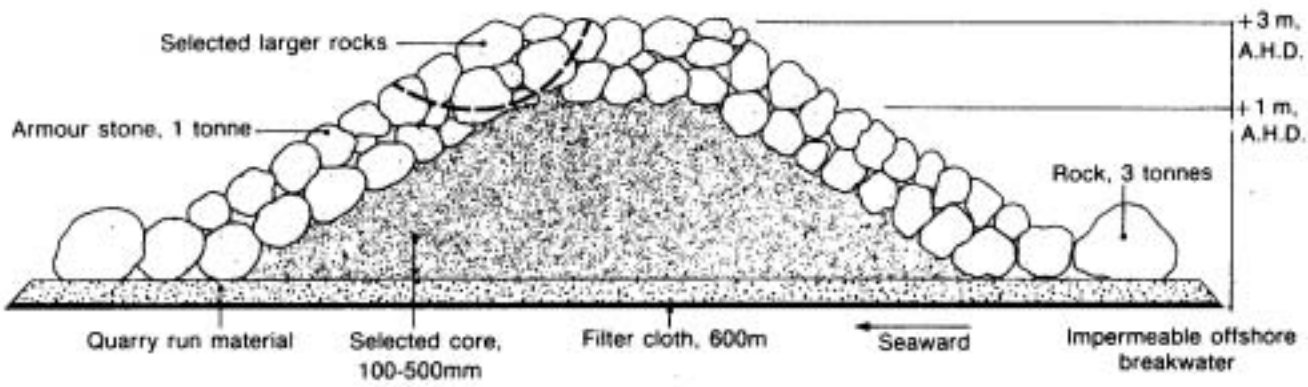
Like groynes, offshore breakwaters require a supply of sand, and failing this, merely cause a redistribution of sand along the beaches. As with groynes, they could only be used at Adelaide in combination with beach replenishment.

Methods, Assumptions and Costing

The breakwaters suggested and costed in the Alternatives Study are 300 m long, parallel to the shore, and a distance of approximately 300 m from high water mark. The consultants based the design on a design used at Tel Aviv, Israel (Fried 1976), and on published descriptions of the performance of these and other breakwaters, and related this to their estimates of active beach width (Section 5.7.4). As for the groynes in Alternative 5, the spacing assumed here is greater than that used elsewhere.

The breakwaters were assumed to be impermeable ones, with their top level above water at all tides. The consultants drew attention to some advantages of using lower, permeable, breakwaters (these are thought to better promote a shoreward movement of sand, and cost less to build), but considered that the more conservative assumption should be made at this stage, because of the limited use of this coast protection method to date and the small amount of information on it. The suggested design, as shown in Figure 59, includes a bottom filter to reduce settlement, and a core of selected material. The 1 tonne armour rock suggested by the consultants is light in comparison to the 3 tonne rock use on the seawalls and would need to be reviewed. The consultants' cost estimates are used here without alteration.

FIGURE 59 OFFSHORE BREAKWATER DESIGN, CROSS-SECTION (KINHILL STEARNS AND RIEDEL & BYRNE 1983)

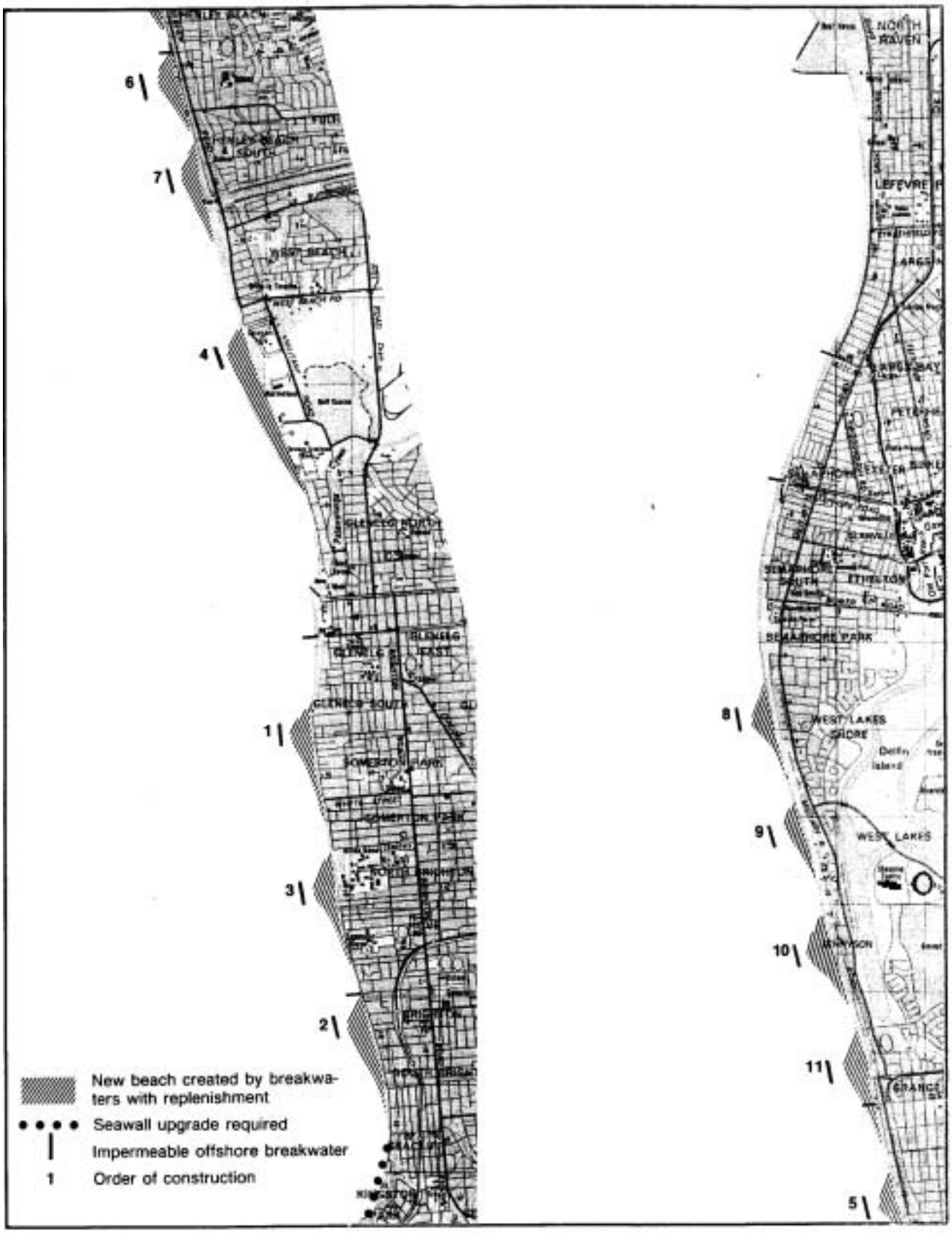


The relatively large tidal range at Adelaide, as compared to places where offshore breakwaters have been used, is a significant disadvantage, because it necessitates higher structures and impedes construction. It contributes to making this method very costly.

The breakwater spacing and construction sequence suggested by the consultants is identical to that adopted for the 'groyne' strategy and is illustrated in Figure 60.

At least two construction methods are possible – either to build an embankment out from land and to remove this after construction, or to construct from floating platforms, possibly supported on spuds. The consultants assumed the latter for their costing, and assumed that materials would be loaded onto barges either across the beaches or from somewhere in the southern part of the study area. Either Port Stanvac or the new boating facility at O'Sullivan Beach could possibly be used with appropriate modifications. Loading could be possible at Glenelg if a channel were dredged, and if fairly small barges were used, though tidal constraints could be expected to affect the costs.

FIGURE 60 ALTERNATIVE 6, MAJOR REPLENISHMENT WITH OFFSHORE BREAKWATERS (KINHILL STEARNS AND RIEDEL & BYRNE 1983)



Loading at Outer Harbor was not assumed, because of the extra transport distance. Detailed consideration of the methods was beyond the scope of the Alternatives Study, and the estimated costs should be considered only as probable order of costs covering a range of possible construction methods.

Because of the local novelty of the construction methods and the small amount of overseas experience, the consultants had difficulty in estimating costs for the breakwaters, and this should be borne in mind in comparing the cost of alternatives. The costs for the breakwaters were estimated between $821,000 and $1,094,000 each depending on how far the materials had to be transported. The construction timing was assumed to be as for the groynes of Alternative 5, with the first six being built in the first 10 years and the remaining five between years 10 and 20.

Other costs for this strategy, as set out in the present day value calculation in Appendix A, are identical to those assumed for Alternative 5 (Replenishment with Groynes) and explained in Section 5.7.5. The same amount of additional initial replenishment is assumed to allow for the uneven distribution caused by the structures, and for the sand 'locked up' behind them not being available at other places.

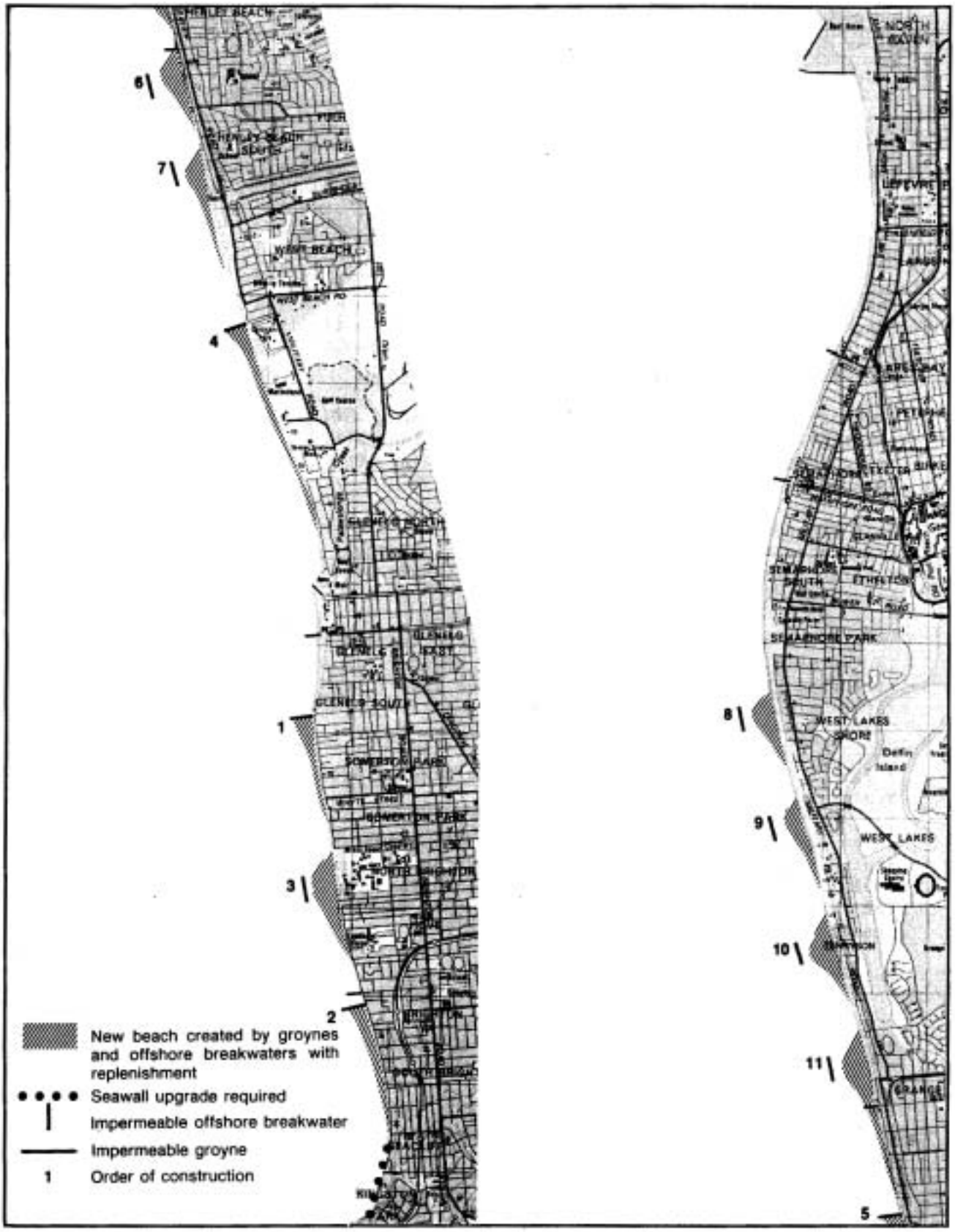
Advantages

- This strategy has the same advantages as 'groynes with replenishment' in reducing the need for sand trucking to compensate for alongshore losses, in reducing dredging costs at the Patawalonga channel, and in avoiding maintenance at stormwater drains.
- As with groynes, most of the present problem areas could be protected (perhaps more positively than with replenishment alone), although at the cost of increasing the erosion risk at other places.
- Compared to groynes, the erosional side effects are less harsh, though the effects are similar, especially on the downdrift coast.
- Sand losses offshore may be less than with groynes.
- The breakwaters would have less impact than groynes on the beach appearance and amenity, and on safety of swimmers. The varied width of beaches would cater well for boating and for the establishment of new grassed picnic areas on the accumulation areas behind the breakwaters. A variety of swimming conditions would be available, with calmer conditions behind the breakwaters, and wave conditions between them being similar to those at present.

Disadvantages

- At a present day cost value of $27.6m (5% discount rate), this is the most costly of all the strategies considered in this review. It would cost approximately $3m more than the replenishment with groynes option, compared with which it has few advantages and more uncertainties.
- The performance of the breakwaters is even more uncertain than that of groynes, not only because of the variable wind and wave conditions, but also because the effects of the tidal range, which exceeds that at other offshore breakwaters, are largely unknown.
- The strategy is subject to the same uncertainties, risks and possible additional costs (eg for new seawalls or additional structures to counter problems that may arise) as for use of groynes.
- The effect of the breakwaters on nearshore seagrasses would be greater than that of groynes, but the consultants for the Alternatives Study considered that this would still be small compared to the losses occurring at present, and compared to possible losses associated with a major replenishment. These are discussed in Section 5.7.4.
- Because of the large tidal range, the breakwaters would be large structures, which would be very significant visual barriers.
- There would be an increased hazard to boating, especially to sailing dinghies, which are used in large numbers on weekends.

FIGURE 61 ALTERNATIVE 7, HYBRID SOLUTION (KINHILL STEARNS AND RIEDEL & BYRNE 1983)



5.7.7 Hybrid Solution (Alternative 7)

This strategy is similar to the two previous ones except that it includes both groynes and breakwaters, with these being located to minimise costs and to use the characteristics of each to the best advantage. A short length of seawall upgrading at Seacliff is included, as for the other two options.

The consultants' suggested arrangement is shown in Figure 61. Offshore breakwaters are mainly used to minimise effects on the beach amenity and to provide a less harsh effect on the coast at Henley and north to West Lakes, where any erosion induced by the structures would have serious consequences. Groynes are used where a more definite effect is required, such as at The Broadway (to avoid silting of the Patawalonga channel), and to reduce costs where downdrift erosion is less critical and can be accepted.

Methods, Assumptions and Costing

The methods are similar to alternatives 5 and 6, with the same assumption that all of the structures would be built in the first 20 years. The suggested sequence is shown in Figure 61 and the detailed expenditure assumptions are shown in Appendix A.

At a present day cost value of $27.0 million (5% discount rate), this strategy would cost marginally less that Alternative 6 (Major Replenishment with Offshore Breakwaters), and offers only very slight advantages.

With the exception of the slight cost saving and the more positive effect for the Patawalonga channel, its advantages and disadvantages are similar to those for the groyne and offshore breakwater alternatives, mainly the latter.

To avoid unnecessary discussion, this alternative is not carried forward to the main comparison of strategies. However, it should be borne in mind that groynes or breakwaters can be substituted in either of alternatives 5 or 6, with appropriate cost adjustments.

5.7.8 Other Combinations of Groynes/Breakwaters with Beach Replenishment

There are many possible combinations, not all of which can be considered separately here. One, however, warrants special mention – this is to replenish between Brighton and West Beach (as for alternatives 4 to 7), and to provide groynes or breakwaters only to the coast south of the present Patawalonga groyne.

This may seem attractive at first glance as a compromise that avoids the undesirable and risky use of groynes in the Grange/West Lakes area and that seems to place greater emphasis on the southern part of the study area where the erosion problems originated. However, the main advantage of using groynes or breakwaters is largely lost. The full sand top-up requirement would then have to be provided at North Glenelg (perhaps reduced by the small amount that passes the structures and that would consequently be needed at Brighton), and the costs and nuisance of trucking would continue.

The only advantage would be to reduce maintenance dredging at the Patawalonga, though the cost of doing this would exceed the saving. The level of protection afforded to the coast between Seacliff and Glenelg might be lower and the risks rather higher than if replenishment were used alone.

This alternative has therefore not been costed or carried forward for comparison with the others.

5.8 A TRIAL GROYNE AT THE BROADWAY

The Alternatives Study recommended that an experimental groyne be built at The Broadway both to determine the behaviour of the groyne system considered in Section 5.7.5 and for the direct benefits that it was seen to offer. This recommendation was in the context of a major beach

replenishment. It is discussed here both in that context and for a continuation of the present measures. The groyne would be an impermeable one to the design shown in 5.7.5, approximately 200 m long, and would be located at the site of the present small groyne.

The main information the groyne would provide would be on the alignment of the beach created and consequently on the length of the 'tail' of the fillet of accumulated sand, and the seasonal variation in this. Special conditions apply at the existing structures at Glenelg, North Haven and Outer Harbor, making these an unreliable guide to the behaviour of groynes at other parts of the metropolitan coast. A trial groyne would also provide additional information on the rate of alongshore sand movement. To be useful for this it would need to be a totally occluding structure. The information obtained could be valuable for replenishment strategies as well as for structural ones. However, as discussed in Section 2.5.4, a structure such as that proposed would not necessarily provide good alongshore transport information, because of its sheltering of wave energy from the north-western sector, and because sand transport may be rapid and obscured by seasonal effects. Depending on climatic conditions, it could be as long as 10 years before reliable information could be obtained from the trial or, alternatively, it could be filled very quickly. The Patawalonga groyne, built in 1964, filled very slowly at first, and results over the first year or two would have been misleading. However, it then filled very rapidly. It has provided very little useful information on rates of sediment transport.

A groyne at The Broadway would give an improved beach for perhaps as much as a kilometre to the south (this beach is presently depleted and has received little apparent benefit from the beach replenishment program). It would also reduce the amount of sand reaching the Patawalonga, and would facilitate and reduce the cost of dredging and maintaining a boating channel. The location is a good one because the coast immediately to the north, between The Broadway and Pier Street, is already without a beach at most times and has a reasonable seawall to withstand any additional erosion effects of the groyne. The structure would be updrift of the existing Glenelg groyne, which would limit the downdrift erosion, though the natural bypassing of the smaller existing structure would be reduced, with effects further north.

Construction of a groyne at The Broadway does not have to be a first stage of a 'groyne' strategy. A single groyne at this location could be included either in the present strategy or in a major beach replenishment strategy, though it does affect regular and top-up replenishment quantities.

The important question is whether the advantages and the information to be gained would be worth the cost (both initial and flow-on) or the possible consequences; and, if the groyne is to be built, whether this should depend on whether or not a decision is made to do a major beach replenishment using offshore sand. Before considering these it is useful to consider the groyne's likely effects and the consequences more fully.

- As noted, sand would accumulate on the south side of the structure, providing improved beaches and better coast protection for a certain distance to the south. This accumulation would provide valuable information for the design of additional structures, should these be proposed, and would provide some (probably less reliable) information towards design of beach replenishment schemes.
- The sand supply to the Glenelg beach would be markedly reduced, at least for the first few years, with a lesser but significant effect thereafter. The main Glenelg beach would narrow, but would remain a good recreational beach. The already poor beach between The Broadway and Pier Street could be expected to disappear almost entirely, leaving perhaps a small fillet against the new groyne. The consultants considered that the reduction of energy from the south might induce more sand to move southwards from the main Glenelg beach to form a beach near Pier Street.
- After a few years the natural bypassing of the Patawalonga groyne would reduce or cease. This would make dredging and maintaining a boating channel easier and less costly, but it would mean that the regular replenishment to North Glenelg would have to be increased. The extent of this depends on how the present, poorly understood, bypassing is affected.

- The sand carting program would thus need to be increased above its present level. However, the present level has, for the purpose of this review, been assumed to be the maximum that can reasonably be achieved with trucking.
- Sand accumulated at the new groyne would add to the general sand deficiency at other places. The groyne could be filled artificially by trucking (assuming no major replenishment), but this would deplete scarce sand sources and would reduce the experimental value. It could also not be achieved without either additional trucking or reducing the present program.
- As discussed under the 'groynes' alternative (Section 5.7.5), groynes may increase the loss of sand offshore, though the effect of a single groyne would not be large.
- The single groyne at this location would have little impact on the beach amenity other than by providing an improved beach to the south at the expense of the loss of a lesser length of poor beach.
- From a cost point of view, the Patawalonga dredging advantage, discussed in Section 5.5.2, would seem to be achievable at the cost of only one groyne (approximately $0.5m) rather than a whole groyne strategy. This is offset by the cost of the additional regular replenishment or top-up (for the major replenishment alternative). However, there could still be a slight cost advantage, though this depends on the channel dredging proceeding.

On balance, the construction of the groyne would not seem to be justified, though this should perhaps be reviewed if a major replenishment strategy were to be adopted and if a firm decision was made to dredge and maintain a boating channel at the Patawalonga. The 'groynes with replenishment' strategy is not recommended in the conclusions of this review, despite the support it received in the Alternatives Study, and information relating to the design of groynes is thus not imperative. In addition, the protection advantages do not appear to be sufficient to warrant the required increase in sand trucking, which would be very difficult to achieve.

5.9 ENVIRONMENTAL AND SOCIAL CONSEQUENCES

The Alternatives Study considered the environmental and social consequences of the various strategies. The findings will be discussed briefly here, with the inclusion of a few general observations made by the Branch and other organisations.

No further work has been done by the Branch, and the quantitative evaluations made in the Alternatives Study have only been altered to take into account major changes in the assumptions, such as for the extra beach replenishment quantities included in the groyne and breakwater alternatives. These changes do not alter the tenor of the consultants' comparisons of the alternatives on environmental and social grounds. It was not considered worth changing the consultants' figures to take account of the lesser changes in the volumes of construction materials or the transport of these, as the effect on the environmental assessment would not have been significant.

The Alternatives Study addressed the following environmental and social issues:

- general beach amenity;
- boating facilities;
- seagrass beds; and
- temporary effects of construction and maintenance operations.

5.9.1 General Beach Amenity

Beach Losses and Gains

The consultants' monetary valuation of beach losses and gains is described in sections 3.4 (Beach Loss) and 5.5.1 (Recreational Value of Beaches) and set out in Table 8 under the latter heading. These values can be assumed to reflect reasonably accurately the gains and losses in general

beach amenity. The losses that would occur if no action was taken, or if a 'seawalls only' strategy were adopted, would be considerable, to the detriment of public enjoyment of the beach and of tourism to the state.

For both these alternatives, the changes to the southern beaches would occur progressively with substantial beach loss being evident in 20 years time, and being severe by the end of the 50-year period considered in this review. In the southern area only a few isolated beaches would remain – at Glenelg, Minda and West Beach Trust for the 'no action' alternative. For the 'seawalls only' alternative, the beach at Glenelg might be the only one that would remain in this southern area. There would also be notable beach losses at Henley and Grange for both of these alternatives. The consultants estimated that about 80% of the total number of beach users recreate on those parts of the beach that would be lost.

If the present strategy of seawalls with replenishment were continued, the beaches would progressively improve, with more beaches becoming available at high tide, eventual development of dunes behind most of the beaches, and consequently less winter draw-down of beaches. The values for this option in Table 8 reflect the increased beach amenity.

For all the major replenishment options, beach improvement would be effected more rapidly, with the new wide beaches providing more space for beach recreation.

Effects of Groynes and Breakwaters

The comparison of beach amenity for the alternatives that combine a major replenishment with groynes or offshore breakwaters is difficult, because of the subjective elements involved and the different ideas that people have about what constitutes a 'good' beach. The consultants considered that the variety in beach type and width resulting from the use of structures would add to the beach's attraction and that groynes would not be aesthetically displeasing, because of their wide spacing. As noted in Section 5.3.1, groynes have added to the popularity of beaches elsewhere. However, this might not be as applicable at Adelaide beaches because fishing platforms already exist at the jetties along the metropolitan coast. The groynes would nevertheless provide useful windbreaks.

The Department of Tourism and the Department of Recreation and Sport, in commenting on the Alternatives Report, have both expressed concern about the effect of groynes on beach recreation and on tourism, on grounds of appearance and because of the impediment that they would present to walking or jogging along the beach. However, this impediment would not be severe at the spacing proposed, but would certainly become more significant if additional groynes were found to be necessary later and had to be inserted between those proposed. The loss of beach continuity caused by erosion on the downdrift side of each groyne would be likely to impede pedestrian movement along the beach.

Groynes would be likely to create a more dangerous swimming beach, with deep water closer to the shore in places and with seaward currents caused by the groynes. The Coast Protection Board particularly drew attention to this when it considered the Alternatives Report.

The offshore breakwaters of Alternative 6 would not themselves disrupt the beach continuity, but erosion of the beaches between the breakwaters would be likely to result in portions of beach being lost. Offshore breakwaters would provide areas of even safer swimming beach than exist at present, without introducing swimming hazards elsewhere.

Loss of Northern Beaches

The consultants did not consider the long-term effect that use of either groynes or offshore breakwaters would have on the beaches north of West Lakes. Because of the wide beaches in the Port Malcolm and Semaphore area, the erosional effect would be unlikely to become serious until perhaps 20 years or so after construction of the most northerly groyne or breakwater – ie in 30 to 40 years time. Severe loss of the Semaphore beach and the reserves backing it could be expected at that time, with a consequent loss of amenity to residents of that area and to other users of this

beach. This could be avoided by a localised beach replenishment from the nearby North Haven accumulation, should this prove necessary.

5.9.2 Boating Facilities

Boating is the most important beach use after swimming, sunbathing and walking. It is dependent on changes to the beach, especially where these affect vehicular access or result in damage to boat ramps.

The effect of beach loss on boat ramps has been discussed in Section 3.2.2, where it was noted that the main concrete ramps at North Haven and Glenelg would not be greatly affected by loss of beaches, but that some of the smaller and more exposed ramps would need to be rebuilt when damaged, and that the greatest effect might be the loss of beach space for rigging and launching boats, especially sailing dinghies. This beach loss would apply for both the 'no action' and 'seawalls only' alternatives, commencing in 5 to 10 years time, and becoming more critical in 20 to 30 years time.

All the replenishment alternatives, including continuing the present measures, would provide more beach space for rigging and handling boats, but would make access onto the beach more difficult. Access arrangements can and would be devised to ensure that boats could be towed onto the beaches at most of the places where this is done now, but more maintenance would be needed. Access might be more difficult despite measures that would be taken. The best method, noted also in the Alternatives Report, would probably be to make short roadways over the new dunes using the white 'lime grit' currently used for some of the ramps. This material does not impair the beaches when eroded onto them. This might be combined with 'beam and chain' type ramps across the soft part of the beach.

Both alternatives 5 and 6 (Major Replenishment and Groynes and Breakwaters, respectively) would provide adequate beach space, but some ramps might need to be relocated, strengthened or extended, depending on their position in relation to the groynes or breakwaters. The breakwaters themselves might cause some restriction to movements of yachts and windsurfers, and could be hazardous to these.

5.9.3 Seagrass Beds

Comment here is limited to the effects that the strategies would have on seagrasses, and does not include the seagrass loss that is presently occurring, or the reasons for this loss. This is discussed in Section 2.7. However, the effects of the strategies need to be considered in relation to this large ongoing loss, and the possibility of initiating a similar instability further south should not be ignored.

Seagrass losses are likely to occur for a major replenishment – at the dredged area, possibly at delivery onto the beach, and possibly during re-adjustment of the beach profile to a more seaward position – and for groynes or offshore breakwaters, some of which would be wholly or partly constructed in seagrass beds.

For a Major Replenishment

The consultants noted that seagrass damage would not be an issue at either the assumed Onkaparinga offshore sand source or for the partly proven and assumed Outer Harbor deposit, because there were no seagrass beds in the actual borrow areas. However, this might need further consideration depending on the area of the eventual sites and their exact location, assuming that sufficient suitable sand can be found at either place. Additional sand to be sought at Outer Harbor would desirably be in deeper water where there may be some seagrasses.

The consultants noted that there were extensive seagrass beds in the previously considered area 'B' off North Haven and that dredging over the large area that would be necessary there would be environmentally undesirable. This sand deposit has since been found to be unsuitable, though further sampling needs to be done before it is totally dismissed. At this stage, its use seems to be most unlikely.

Assuming that a major replenishment were adopted, the two most likely methods of delivering the sand to the beach would be by an onshore pipeline or by pipeline from a hopper dredge moored off the beach. Neither of these would result in damage to the seagrasses, other than that which might be associated with the pipeline to sea and moving it to new positions. The temporary pipeline would be laid on the seabed. This was not commented on by the consultants, but effects would be likely to be temporary and not serious.

The Dredging Study considered other methods of getting the sand ashore – by dropping it in pre-dredged holes and rehandling it, and by barging it in close to the beach and dumping it, allowing it then to come ashore by natural processes. The first method is unlikely because of the cost (see Section 5.2), and the second is too uncertain to risk using – having regard to the trial dumping carried out and described in Section 2.5.4. The consultants considered that the greatest seagrass damage could result from either of these methods and that, if the latter were adopted, care should be taken to ensure that the sand was dumped inshore of the seagrasses. The experiment has shown that this would not be practical and, although most of the tracer sand is thought to have moved shorewards, the results are not conclusive. Although unlikely to be relevant, this is noted here in case either of these methods should be considered in the future.

Another possible cause of damage not addressed in the Alternatives Study is through initial spreading of the sand seawards as the new beach adjusts to an equilibrium profile. Although the beach would be reverting to a condition closer to its original one than applies at present, the change might be rapid and beyond the capacity of the nearshore seagrasses to adapt to. There is a risk that this could initiate damage similar to that already occurring further north. There is no evidence of seagrass loss associated with the beach replenishment to date, and the Branch is not aware of the problem having arisen elsewhere. Any effect could be minimised by spreading the replenishment as much as practical, or by doing it more slowly.

For Groynes or Breakwaters

Breakwaters or groynes north of Glenelg would be inshore of the seagrasses and only the three structures (groynes or breakwaters, depending on the alternative) south of Glenelg would be either wholly or partly in the seagrasses. The consultants estimated that the area of seagrasses affected by groynes would be 2 hectares and that by breakwaters, 15 hectares. They noted that there would be scouring and deterioration of seagrasses in the vicinity of the structures, but did not comment on the possibility that the initial losses could lead to more widespread losses, as are occurring further north. Although this is possible, it is less likely in this southern part of the study area because of the thinner layer of sediment. As noted in Section 2.7, expansion of the eroded areas by sand moving onto adjacent areas seems to be more prevalent where the sediment layer is thicker.

5.9.4 Impacts from Construction and Maintenance Operations

The consultants identified potential impacts of the various strategies at material sources, at the beach and en route, though the nature of many of these was such that they could not be readily quantified or compared. For example, noise is a potential impact at the source beach or quarry, en route, and at the destination beach or sea defence construction site. Similarly, sand nuisance can occur at the source, en route and at the destination beach, and public recreation activities and pedestrian movements can be disrupted at all three locations. Accidents could occur at all three, but disturbance to the visual and recreational amenity would only occur at material sources and at destination beaches. The consultants noted that the main physical environmental impacts would be likely to be to seagrasses at a marine sand source.

The onshore sand source at Torrens Island has since been discovered. Its use, which would only be required for Alternative 2 (Continuing the Present Measures), would cause considerable alteration of the natural environment on Torrens Island. Against this, part of the area has already been used for dumping and sand mining, and the whole would be likely to be used by others if not used for beach replenishment. It is unlikely that the area would be as well rehabilitated after small-scale private mining as with a planned operation by the Board.

The consultants adopted a broad comparison method – by summing the material quantities and truck-kilometres of transport required for each alternative, and then comparing these. Their results, with alterations to take account of the major changes in the assumptions for some alternatives, are set out in Table 13. It must be stressed that this table is not strictly consistent with all the recent assumptions, but that the discrepancies are small and do not affect the comparison.

TABLE 13 COMPARISON OF CONSTRUCTION AND MAINTENANCE IMPACTS OVER 50-YEAR PERIOD

No.	Alternative	Total Material Quantity* (m^3)	Truck Travel** (truck-km)
1	No protection	Nil	Nil
2	Continuation of present policies	4,600,000	28,200,000
3	Seawalls without replenishment	380,000	1,000,000
4	Major sand replenishment	4,600,000	10,700,000[tt]
5	Major sand replenishment with groynes	3,800,000[t]	2,600,000
6	Major sand replenishment with off-shore breakers	4,000,000[t]	3,300,000
7	Hybrid solution	3,900,000[t]	3,100,000

* Includes dredged sand, trucked sand and trucked rock with quantities indicating in place amounts.

** Includes truck travel involved in sand and rock carting.

Table based on Table 6.2 of Kinhill Stearns and Riedel & Byrne (1983), with the following alterations:

[t] Increased to take account of the 350,000 m^3 of additional sand considered necessary for these alternatives.

[tt] Increased to include top-up replenishment for years 10 to 20, consistent with the assumption here that the initial major replenishment would be done rapidly rather than over a 10-year period, and would not include the years 10 to 20 top-up.

This table shows that material quantities are very similar for all the viable options. Those alternatives that combine groynes or breakwaters with a major replenishment require rock and other materials, which approximately compensate for their saving in sand requirements. The 'seawalls only' option clearly has much less impact from construction and maintenance activities, but this is offset by loss of the beaches.

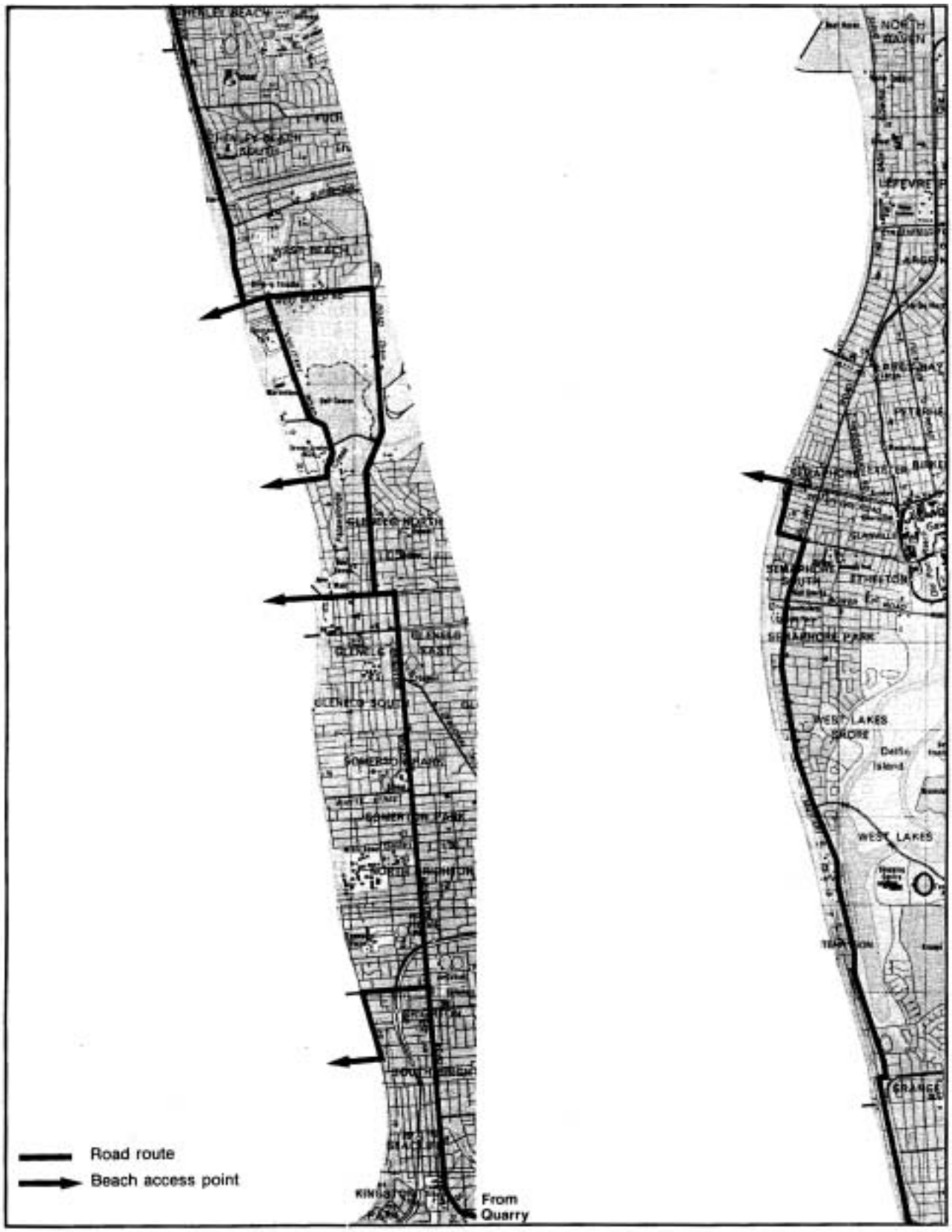
Continuing the present measures involves nearly three times the trucking as the top-up replenishment for a major replenishment, and this is almost three times as much as for replenishment with groynes or breakwaters.

Truck Routes

The Alternatives Study considered truck routes for sand and other materials, but was unable to suggest improvements on those used at present, as shown in Figure 62. Routes follow roads maintained by the Highways Department wherever practical, and council agreement is sought for other roads. These are usually at the beginnings and ends of routes.

Routes are varied as much as possible within the constraints imposed by the necessary programs and the available access points. Unfortunately, there is little scope for variation, and this means that small groups of residents are repeatedly subjected to truck movements past their houses.

FIGURE 62 MINIMUM IMPACT TRUCK ROUTES (KINHILL STEARNS AND RIEDEL & BYRNE 1983)



Minimising Sand Carting Impacts

The consultants considered this also, but were again unable to suggest improvements that had not already been considered by the Branch and applied where useful or practical. The following possibilities have been considered by both the consultants and the Branch:

- **Removal of Sand from a Beach.** Wherever practical, sand is removed in thin layers from the wet part of the beach in the active beach zone. This enables the quickest beach recovery. It also avoids the nuisance of blowing sand on the beach or from the trucks.
- **Avoiding Sand on the Roads.** The sand is transported moist, usually after some draining from small stockpiles (sand direct from the tidal zone is too wet and this would cause a nuisance). The contracts allow for the use of tarpaulins to cover the sand in windy conditions, but this has not been found necessary. Very little sand escapes en route. The problem is mainly where the trucks go on and off the beaches carrying wet sand on their tyres. Regular sweeping of the roads at these places has been found the most practical way to deal with this. Washing the tyres has been tried but was found not practical.
- **Depositing Sand in Thin Layers on the Beaches.** This is generally done except when high tides and stormy conditions are anticipated, when it becomes unnecessary because of the effect that these have in distributing the sand. Sometimes situations do arise when tides and space on the beach preclude even deposition, but this is rare and avoided when possible. Working on the beach can be very difficult at times, with expensive machinery being at risk, and practical considerations must sometimes take priority if the project is to proceed.
- **Use of Elevating Scrapers.** These machines can scrape up the beach sand in thin layers and travel along the beach to the replenishment site, where they can spread the sand as they unload while travelling. They have previously been used for moving sand from the Torrens Outlet to the West Beach dunes, and would probably be used for the recommended bypassing of the Torrens Outlet. Although their use would avoid trucking in residential streets, they are economically out of the question for the main replenishment program. They are also undesirable from beach amenity and safety viewpoints, and could be impractical for much of the coast except at low tide. Judicious use of scrapers to redistribute sand brought in by trucks could avoid some truck movements, but the effect would be slight, and beaches would need to be closed for public safety.
- **Restrictions in Times and Dates of Operation.** It has so far been necessary to carry out most of the replenishment between spring and autumn, though summer holidays have been avoided. This is because of difficulties in running trucks on the beaches, especially the replenishment ones, during the winter months, when tides are generally higher and less predictable. However, winter operations would be better because there would be less nuisance to beach users and because beaches would be smoothed out more quickly in the stormier conditions. The Board has recently built a ramp onto the Brighton beach to give access onto this part of the beach and to enable winter operation, which will be tried in 1984. Depending on the success of this, other methods of getting the sand onto the beach during winter may need to be tried.

  The contracts to date have generally stipulated that work should not be carried out before 7.00 am or after 6.00 pm. Further restriction in the mornings and evenings is possible, but would be costly, because of restrictions also imposed by the tides.
- **Better Clean-up After Contracts.** Obviously, this is no place for economy. Contract clean-up conditions need to be strict and enforced. However, there is a limit to what can or should be achieved, bearing in mind costs and the changes that take place to beaches quite independent of the sand carting, but nevertheless often blamed on it.
- **Safety on Beaches.** Trucks are presently constrained to a maximum of 25 km/h, though this is difficult to enforce rigidly at times when beaches are so soft that trucks must move faster to avoid becoming bogged. Whatever the speed, there will always be some risk unless the particular length of beach is closed to the public while machinery is on it. This deserves serious consideration, though would be unpopular with beach users. The bypass of the Torrens Outlet recommended for most of the alternatives would probably be done using scrapers along the

beach. This is a more hazardous operation, and the beach should be closed during the hours of working.

- **Avoiding Truck Noise.** The worse noise is caused by empty trucks rattling along badly maintained road surfaces. This can be minimised by keeping the roads in a good state of repair with the cooperation of the councils and the Highways Department. Other truck noise can be minimised by setting lower speed limits than the general urban 60 km/h, but this is really only practical for short lengths of road at the beginning and ends of the routes, and is virtually impossible to police. It is probably not warranted.
- **Informing the Public.** There has previously been no advertisement of beach closure, or of operation of equipment on beaches – other than by signs at the beaches themselves. Consideration could be given to a higher level of public information, perhaps by advertisement in the local and daily newspapers, so that the public can better plan their beach trips to avoid the trucking. Conventional advertisement might not be of great value because few people could be expected to read the advertisements or to think of doing so before making a trip to the beach. Greater value could be achieved if the press and radio could be persuaded to provide the information as a public service, in a more prominent way.

Whatever measures are taken, some truck impacts are unavoidable and need to be accepted if the present measures are to be continued, or later if a major replenishment is chosen.

5.9.5 Environmental and Social Conclusions

1. Continuation of the present measures involves the greatest impacts through sand trucking. Some impacts will be unavoidable despite all possible measures. There would be a continued nuisance of machinery on the beaches, temporary beach disfigurement and some hazard (depending on measures to prevent public access to beaches where machinery is working). There would be some alteration of a small dunal environment on Torrens Island through use of this sand, but the effect could be less than might otherwise occur if the sand were used for other purposes.

   These negative impacts would be offset by a gradual improvement in the beaches, providing more recreational space as well as a more natural beach appearance.

2. A major replenishment would provide the better beaches more quickly, and would enable trucking to be deferred for 10 to 20 years, when it would be needed for top-up to replace losses. Trucking would be much less than with continuing the present policies. Depending on the sand source and the method used, the replenishment operation itself would have little impact on coastal residents and beach users after the first few years. During these first few years, there would be considerable activity on the beach with the main beach impact being the shaping of new dunes using earthmoving equipment.

   Vehicular beach access for launching boats may become more difficult, despite measures that would be adopted to facilitate access across the new dunes and wider beaches.

   Marine damage is possible (the seagrass beds would be the main issue) depending on the sand source and possibly on the method of getting the sand ashore, though the latter would be unlikely to present problems. There is also a possibility of seagrass damage caused by a rapid seaward movement of the coastline.

3. The use of groynes or offshore breakwaters with replenishment would have only minor direct effects on seagrasses (in addition to possible effects of the replenishment), but there remains uncertainty about possible flow-on effects following an initial disturbance. There would be an altered beach appearance, with the effect of the groynes being more noticeable than that of breakwaters. There would be some beach discontinuity for both, though the effect of groynes would be greater than that of breakwaters. The beach shape would be altered and there would be an effect on beach amenity, but any assessment of this remains subjective and debatable.

4. The options of taking no action or of building seawalls and not replenishing beaches would both result in significant loss of beaches and would reduce a major recreational amenity. The effect on tourism would be significant.

5.10 COMPARISON OF ALTERNATIVES

This comparison addresses only the most important points. The reader is referred to sections 5.7 and 5.9 for more detailed supporting information on each alternative, and for a more complete discussion of the relevance of the aspects covered here.

The construction and maintenance costs for the various alternative strategies are set out in Table 10, with the calculated present day values for various discount rates. The cost assumptions are explained in Section 5.7 for each alternative and the annual expenditures set out and summed in Appendix A. The concept of present day value comparison is described in Section 5.1.3, where it is concluded that a 5% rate is the most applicable. Except where otherwise stated, reference to costs continue to be to present day values at a 5% discount rate. Where other discount rates are discussed or where clarity is needed, an abbreviation of the form 'PDV 5%' is used.

The variation of present day values of construction and maintenance costs with discount rate is shown graphically in Figure 63. This figure, Table 10, and the earlier tables 8 and 9 (Values of Beach Gains and Losses, and Patawalonga Maintenance Dredging Costs) are reproduced in Appendix B for more convenient reference.

The present day values of costs, benefits and losses are deliberately not combined here, as was done in the Alternatives Study. This is because a direct comparison of total present day benefits is considered to be misleading in the circumstances and to distract attention away from more critical aspects such as the risks involved with a particular alternative, or whether or not it may be suitable at all. The difference in reliability of the beach benefit costs compared to the construction and maintenance ones also argues against their summation. Similarly, the inclusion of a hypothetical Patawalonga maintenance dredging cost in final comparative figures does not seem to be justified. This is not to downgrade the importance of these factors, but rather to ensure that they are considered in their proper context and that the comparison is not oversimplified.

It should be noted that the costs here differ from those in the Alternatives Study, mainly because of changes in the assumptions about onshore and offshore sand deposits, timing of dredging, and of replenishment sand quantities needed for the groyne and breakwater alternatives. Other changes have influenced them to a lesser degree. A difference in the method of calculating present day values accounts for a further amount by which the costs here exceed those in the Alternatives Study. Cost comparisons are valid within each of the two reports but should not be made across them.

5.10.1 Protection Provided, Risks and Uncertainties

Design Criteria

The usual engineering methods of balancing a design against the maintenance costs due to minor failures and against the probability of major failure for an event of a particular return period cannot be applied rigorously here. This is because it is very difficult to define a storm event of a particular return period or to predict its effects on the coast. Notwithstanding this, the concept underlies the design and costing of alternatives and much of the discussion here. Damage to sand dune fencing, walkways and some boat ramps could be accepted as regular annual events, or perhaps slightly less often than this. Minor damage to seawalls, costing perhaps $30,000 to $100,000 to repair, could be considered acceptable once every 5 years or so; damage between $100,000 and $1m could be tolerated once every 10 to 20 years; and damage in excess of $1m might be considered unacceptable at any time. Designing to prevent all damage would be wasteful and uneconomical.

Another approach, which is probably more useful here, is whether a method provides a logical and practical solution that would work in harmony with the natural beach processes. Alternatively, if this is not possible, risks and side effects caused by interaction with the natural processes must be predictable and acceptable. This approach may provide a better comparison of methods that must work over long periods, and where satisfactory function, which is not necessarily guaranteed, must be given precedence over minimising costs.

As discussed in Section 2.7, seagrasses play an important role in the coast's dynamics, and the mechanisms and reasons for the substantial present seagrass recession are not fully understood. At this stage, it is not possible to take into account the possibility that different strategies might influence, or be influenced by, seagrass changes in ways that would alter their relative effectiveness.

Taking No Action (Alternative 1)

The option of taking no action fails whichever approach is adopted. With the total estimated property loss being approximately $30 million over the 50-year period (approximately $12 million PDV), including damage to seawalls, some action is clearly warranted even if the considerable value of the lost beach is not taken into account. In addition, the strategy allows a continuation of a situation where structures interact with natural processes to cause damaging erosion to adjacent areas and beach drawdown, leading to failure of the seawalls. This is neither logical nor practical in the long term.

Seawalls without Replenishment (Alternative 3)

There is no doubt that property protection can be provided by seawalls without risk of failure, provided that the seawalls are maintained or replaced as they fail through undermining due to the beach loss they cause. The uncertainty is not so much in the method as in the assumptions as to how long the existing seawalls would last and when the eventual extensive replacements would be needed. The majority of recently constructed rock seawalls have been assumed adequate to last the full 50 years with only minor strengthening of some of them. Even if they do survive that long, which is by no means certain, failure is inevitable after the 50-year period.

Like the 'no action' option, this method is basically unsound over a long period. However, it can be made to work, provided that eventual, almost total, beach loss is accepted as a consequence.

The Replenishment Alternatives (Alternatives 2 and 4)

Either continuation of the present measures (mainly trucked replenishment) or a major beach replenishment from offshore would provide a satisfactory level of protection, with a similar high level of protection prevailing at the end of the 50-year design period. The major replenishment would provide this higher level of protection much sooner. Either would be achieved with very little risk of failure, though this assumes that an offshore source is found with sand more suitable than that in the partly proven Outer Harbor deposit.

Although the Outer Harbor sand is marginal because most is slightly too fine, it is unlikely that sand losses would be sufficient to result in 'failure' of the strategy. Losses would be small enough to be replaceable, but at a possible additional cost of the order of $3 million (PDV 5%). Use of the Outer Harbor source should not be considered unless the present surveys show that there is no other more suitable source, and not until a large sand volume has been proved.

A major replenishment would provide quicker alleviation of continuing problems such as erosion at the West Beach and Minda dunes and the loss of beach at Somerton and North Glenelg. The strategy allows for some new seawalls at Henley and for bypassing the Torrens Outlet – to provide protection of that area pending the natural northward spreading of the replenishment sand. The present problems would also be solved by continuing the present measures, but would take longer and would entail an interim (but acceptable) risk of storm damage.

The establishment of dunes and the behaviour of the new beaches (especially the spreading of the new sand offshore and alongshore) remain uncertainties for both the present measures and a

major replenishment. However, both methods are sufficiently flexible to allow for adjustments, such as replenishment locations and quantities, and methods of dune building.

The discharge of stormwater through artificially maintained gaps in the new dunes is probably the greatest uncertainty. If the proposed method were not to work satisfactorily, costly works could be needed to redirect some of the stormwater and to modify outlets.

Both the replenishment options would work satisfactorily regardless of climatic variations and changes in alongshore drift from year to year or in single major events. They have the least risk of failure due to unusual events or weather patterns.

The design of neither option is as sensitive as the groyne and breakwater options to knowledge of alongshore sediment movement or wave energy direction and the effects of these on the coast's alignment. The present sand carting program is more easily adjusted to take account of changes in the transport rate or in the event of the estimate of average transport rate being incorrect. The major replenishment quantities are only influenced to a small degree by the transport rate, the volume mainly being required to allow for spreading across the beach profile and to resist a defined storm event. The top-up part is affected, but most of this is trucked in later years, and can be varied if found desirable.

A reduced version of the present measures – to maintain the status quo only – was considered but rejected. The outstanding problems would remain, with continued erosional interactions between the sea and coastal structures. This option would not cover the uncertainty in the alongshore transport rate and other processes, and could be found to be inadequate if the present estimates of these are found to be incorrect. Since the cost difference over the first 20 years is small compared to a full continuation of present measures, there is very little justification for this option.

Groynes and Offshore Breakwaters with Replenishment (Alternatives 5 and 6)

These methods have a higher level of risk than either the present measures or a major replenishment, especially at the wide alongshore spacings at which the structures have been designed and costed.

It is perhaps not appropriate to speculate about the performance of these methods without noting that the consultants have emphasised, in the Alternatives Report and in discussions on this, that these options depend heavily on performance of experimental, prototype structures, such as the trial groyne suggested for The Broadway. Notwithstanding this, the design seems more likely to be optimistic than pessimistic, and the effects discussed here and in Section 5.7.5 are probable, though some could be avoided by construction of additional groynes, breakwaters or seawalls. The cost implications of this are discussed later.

The structures would cause sand accumulation at some places with uncertain losses at others – depending on fluctuations in wave energy direction. They can thus offer more certain protection at some places, but with an increased risk at other places. The overall level of protection along the metropolitan coast might be higher without the structures than with them.

The main justification for using groynes or breakwaters with replenishment is to reduce the alongshore transport and consequently the trucked top-up sand volumes. However, this can only be achieved with a total commitment to a groyne or breakwater solution – failing this, the top-up is still required but at a different place (updrift of the northernmost structure). This means that the full methods should be considered, including the structures suggested between Henley and West Lakes. The coast between these structures would have very little or no protection to resist likely erosional effects of the structures. It could consequently be at a considerably higher risk than applies at present.

The structures would cause long-term erosion at Semaphore however well they perform, and this would result in serious loss of the beach amenity and threat to the foreshore in 30 to 40 years time. Because they operate by altering the beach alignment and reducing alongshore sand transport, sand movement northward out of the Semaphore area, which would be unaffected by the

structures, would exceed the supply into it, with a net loss and consequent erosion. This could be offset by a local replenishment when this became necessary.

Because of the large tidal range in relation to other places where offshore breakwaters have been used, and because of the general lack of international experience of this method, it would involve more uncertainty than using groynes.

Although the structures may not offer an overall protection advantage (more likely the reverse) they could facilitate stormwater discharge (by arranging for the narrowest beaches to be at the drains). They could also make it easier and less costly to dredge and maintain a channel at the Patawalonga.

As with a major replenishment alone, these alternatives depend on finding an offshore sand source and would be subject to the same uncertainties about loss of replenishment sand if it were too fine.

Summarised Comparison

A major replenishment (alternatives 4 and 4A) would provide the highest level of protection though there could be a risk of sand loss, depending on the sand source, and of additional expenditure to replace the loss.

Continuation of the present measures (Alternative 2) would provide a satisfactory and progressively improving level of protection.

Seawalls alone (Alternative 3) could provide adequate protection of property, but would eventually result in extensive beach loss.

The use of groynes or offshore breakwaters with replenishment (alternatives 5 and 6) cannot be considered viable solutions for Adelaide without prototype experiment. The methods could provide better protection at some places, but would create a greater risk at others, and could result in a generally lower standard of protection than would apply for a major replenishment without structures, or for continuation of the present measures.

5.10.2 Cost Comparison

Construction and Maintenance – Present Day Cost Values at 5% Discount Rate

(Reference should be made to the cost summary tables and graph, Figure 63, reproduced for convenience in Appendix B.)

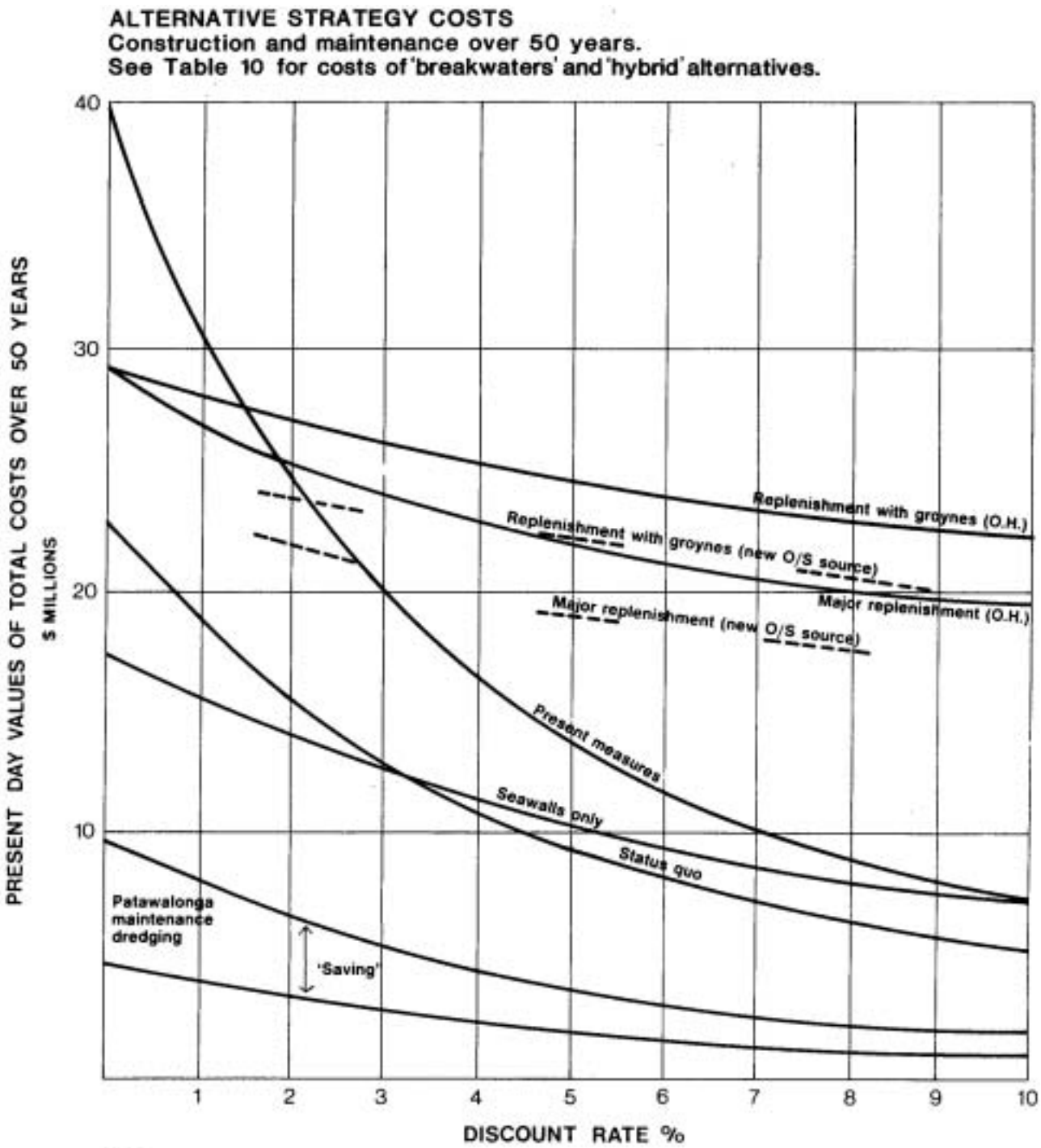
The 'status quo' and 'seawalls only' options are excluded from the cost discussion here. Reasons for exclusion of the 'status quo' option are given in Section 5.10.1. As noted under the next heading, the value of beach loss makes the 'seawalls only' alternative far more costly overall despite its lower construction and maintenance costs.

Alternative 2 (Continuing the Present Measures) has the lowest present day value of the remaining options. At $13.4m its cost value is $5.7m less than that of the next lowest method, a major replenishment from an offshore source yet to be found. Its cost value is $8.2m less than that of doing a major beach replenishment using the partly proven Outer Harbor source. These major replenishment options thus involve cost values 40% to 60% higher than for continuing the present measures. This is despite the high assumed costs of obtaining future sand from scarce onshore sources, though it does assume that the Torrens Island source will be available. The cost advantage over a major replenishment using the Outer Harbor deposit might be even higher (by up to a further $3m PDV), depending on how much of the sand would be lost offshore because of being too fine.

Although this method has the highest actual cost (PDV 0%), comparison without applying a discount rate is pointless. Money does earn interest, and a discount rate must be applied.

The present methods have the lowest cost value by the same amount even when the possible Patawalonga maintenance dredging is taken into account. The dredging advantage only applies to the groynes and breakwater solutions, which reduce the dredging quantities by lowering alongshore sand transport rates.

FIGURE 63 ALTERNATIVE STRATEGY COSTS – PRESENT DAY VALUES OF CONSTRUCTION AND MAINTENANCE COSTS



Note:
Financial risk factors are not included
- for replenishment from Outer Harbor up to $3.0 mill. additional.
- for groynes or breakwaters as above, $3.0 mill. and up to $3.0 mill. for additional structures.

TABLE 14 CONSTRUCTION AND MAINTENANCE COSTS WITH PATAWALONGA MAINTENANCE DREDGING

Present day values at 0%, 2%, 5%, 7.5% and 10% discount rates

No.	Alternative	Present Value of Costs for 50 Years at Discount Rates				
		0%	2%	5%	7.5%	10%
1	No Protection (ie dredging maintenance only)	3.7	2.6	1.7	1.4	1.1
2	Continuation of Present Measures	48.7	30.0	17.0	12.0	9.1
2A	Maintaining the Status Quo	32.9	21.2	12.7	9.4	7.4
3	Seawalls without Replenishment	20.8	16.4	12.1	9.9	8.2
4	Major Beach Replenishment (OH)	38.7	31.0	25.2	22.8	22.2
(4A)	(from new offshore source)	(35.6)	(28.1)	(22.7)	(20.5)	(19.1)
5	Major Replenishment with Groynes (OH)	34.2	29.9	26.2	24.4	23.0
(5A)	(replenishment from new offshore source)	(31.2)	(27.1)	(23.8)	(22.2)	(21.0)
6	Major Replenishment with Offshore Breakwaters (OH)	39.2	34.1	29.4	27.0	25.2
(6A)	(replenishment from new offshore source)	(36.2)	(31.3)	(26.9)	(24.8)	(23.2)
7	Hybrid Solution (OH)	38.8 3	3.5	28.8	26.3	24.5
(7A)	(replenishment from new offshore source)	(35.8)	(30.7)	(26.3)	(24.1)	(22.5)

'OH' indicates use of the partly proven Outer Harbor sand source.

'New Offshore Source' – a suitable source yet to be found.

Alternatives 4 and 4A (Major Beach Replenishment) have the next lowest PDV cost values. At $21.6m and $19.1m respectively for use of the Outer Harbor and assumed offshore sand sources, they are $2.8m (approximately 13%) less than the corresponding cost values for use of groynes with replenishment. This is despite inclusion of the costs for the continued top-up replenishment, which are considerably less for the groynes alternative. The difference reduces to $1.0m (approximately 5%) if savings on future maintenance dredging at the Patawalonga channel are taken into account. A possible extra cost risk of up to $2.5m (PDV) applies to all alternatives that include use of the Outer Harbor sand, because of its fineness. This would be slightly higher for the groyne and breakwater alternatives because of the extra sand volume assumed for these.

When overall financial benefits are compared, taking account of the estimated values of beach gains, the benefit of a major replenishment is close to or may slightly exceed that for continuing the present measures – depending on the sand source and the cost of dredging.

Alternatives 5 and 5A (Groynes with a Major Beach Replenishment) have present day cost values of $24.4m and $22.0m respectively and, as noted, cost approximately $1m more than replenishment alone when a Patawalonga dredging saving is taken into account. The costing of these alternatives has assumed a rather arbitrary additional sand requirement of 350,000 m^3 above that needed for beach replenishment alone. The present day value of the cost of this extra sand is approximately $1m. The cost values for the groyne options are approximately equal to those for the replenishment alone, when reduced by this amount, and taking the Patawalonga saving into account.

The very wide groyne spacing is uncertain and additional costs might need to be incurred in building more groynes or seawalls to prevent or counter possible erosional effects. The 5% PDV of

these (the assumptions are given in Section 5.7.5) is up to $3m. This amount, which applies also to the 'breakwaters' alternative, should be regarded as a cost risk factor.

Alternatives 6 and 6A (Offshore Breakwaters with Beach Replenishment) have the highest present day values for construction and maintenance costs, at $27m and $24.5m respectively for use of the Outer Harbor sand source and an offshore source yet to be found. Their uncertainties and cost risks are of the same order as for the groyne alternative – perhaps slightly higher because of the greater uncertainties about construction methods and performance of the breakwaters.

Beach Value Benefits

(Refer to Table 8, reproduced in Appendix B.)

As discussed in Section 5.5.1, the beach has a considerable value to the community, and the Alternatives Study dollar values of this seem to be of the right order. The effect of the discounting method is to give higher present day values to beach losses or gains that occur soonest, and the choice of discount rate again becomes important. It is arguable that a lower rate should apply to this 'social' or 'amenity' aspect than for the protective, engineering aspects. The higher rates should certainly not apply.

Even at a 5% rate, the present day values of beach loss for the 'no protection' and 'seawalls only' alternatives are $28.4m and $32.8m respectively. This shows that these options are clearly not economical for a comparison of total present day value benefits, even without taking into account the estimated $12m (PDV 5%) property loss for the 'no protection' option.

A major replenishment would provide good beaches sooner, adding much to the beach attraction. The beach value assessment shows a $5.9m (PDV 5%) or an $8.6m (PDV 2%) advantage for a major beach replenishment over continuing the present measures – because the beaches are provided sooner. However, as discussed in Section 5.5.1, there is some doubt as to the validity of extending the beach loss evaluation linearly to beach gains. Nevertheless, whether in perceived or in monetary terms, the improved beaches would at least partly offset the higher costs of a major replenishment, and could in fact justify the choice of this strategy.

The beach benefit evaluation did not distinguish between major beach replenishment with or without groynes or breakwaters.

Sensitivity to Discount Rate

A lower discount rate reduces the economic advantage of deferring expenditure to future years, and tends to favour methods that solve the problems soonest. At a 2% discount rate, which is unlikely ever to be applicable, but which is not impossible, the present day values of continuing the present measures and of a major replenishment are virtually the same (in the range $22m to $25m). The extra cost of using groynes reduces to approximately $2m more than for major replenishment alone, though the Patawalonga dredging advantage increases and the groyne option becomes equal to replenishment alone at a 3% discount rate, and slightly less costly at a 2% rate. This ignores the $3m financial risk factor. Without the 'additional sand', use of groynes has a cost advantage at lower discount rates, from zero to 5% increasing to $2m at 2%.

If 2% is assumed the lowest possible limit, it is therefore apparent that a lower discount rate would reduce the difference between the competitive options to the point where they would become virtually equal. However, for the assumptions made, the cost value hierarchy remains unchanged down to this bottom limit.

Higher discount rates do not have a marked effect on the comparison except, as would be expected, by a quite drastic reduction in the cost value of continuing the present measures – this being the only alternative that involves substantial expenditure in later years. At a 7.5% discount rate, continuing the present measures, at $9.4 m, would have little more than half the present day cost value of the next lowest alternative, a major beach replenishment.

The margin between construction and maintenance costs for major replenishment and for replenishment with groynes reduces very slightly with an increased discount rate. However, the

possible Patawalonga maintenance dredging saving decreases more rapidly, with the overall result being that the use of groynes becomes economically less attractive with increasing discount rate. The trends for offshore breakwaters are similar.

In conclusion, the effect of varying discount rates does not alter the comparison except at very low discount rates, when the present measures, major replenishment and major replenishment with groynes have similar present day values; and at higher rates, when the cost advantage of continuing the present measures increases rapidly.

There is thus no risk of making a wrong economic decision unless major replenishment is chosen for other reasons (accepting its higher present day cost value) and future costs should turn out to be higher than expected. The extra cost of choosing this alternative would then be more than would have been anticipated when the decision was made.

Costs Over the First 20 Years

50 years is an unusually long comparison period, but has had to be used here because of the long-term costs and effects of the methods being compared. 20 years is a more usual comparison period, and the present day values of costs within this period are therefore shown in Table 10.

Comparison over this shorter period obviously favours continuation of the present measures, this being the alternative with the highest expenditure beyond this period. It makes the use of groynes and breakwaters with replenishment seem relatively less attractive, because their costs are incurred in the first 20 years, and their savings (Patawalonga maintenance dredging and reduced top-up replenishment) are only realised after this. This shows why a longer period had to be chosen for the cost comparison.

The small difference between the cost value of full continuation of the present measures and a reduced 'status quo' option is only $1.8m (PDV 5% for the first 20 years), showing that there is relatively little to be gained by making a reduction at this stage.

Cost Conclusions

1. Continuing the present measures, at $13.4m (PDV 5%), has the lowest present day value for construction and maintenance costs, when only those alternatives that provide improved beaches and better protection are considered. It remains the lowest cost option (of those considered here), except at the assumed 2% bottom limit, when the values are virtually equal to those for a major replenishment, with or without groynes. The reduced, 'status quo', version of this strategy has a lower cost – though, on a comparison of overall financial benefit (taking into account beach gains), it rates virtually the same as the full strategy. The cost saving for the status quo option mainly applies in later years and this, together with the other uncertainties associated with this option, argues against its adoption at this stage.
2. The 'seawalls only' option, although having the lowest present day value for construction and maintenance costs, involves extensive beach loss, and the value of this far outweighs its advantage in construction costs.
3. The 'no protection' option has the highest PDV disbenefit of all because of the loss of beaches as well as the property damage that would result.
4. A major beach replenishment, at $19.1m or $21.6m (PDV 5%, and depending on the sand source), has a present cost value $5.7m or $8.2m more than for continuing the present measures. At a 2% discount rate, its present day value is approximately equal to that for continuing the present measures, and at a 7.5% rate is approximately double that for continuing the present measures. A 'risk factor' of up to $3m (PDV 5%) may apply for use of the marginal Outer Harbor sand source. Tests could reduce this figure.

   When overall benefits are compared, taking account of the estimated values of beach gains, the benefit of a major replenishment is close to or may slightly exceed that for continuing the present measures – depending on the sand source and the cost of dredging.

5. The 'groynes with replenishment' alternative, at $22.0m or $24.4m (PDV 5%, and depending on the sand source), remains approximately $2m to $3m higher than for a major replenishment without groynes, reducing slightly with discount rate. The margin reduces to approximately $1.0m at a 5% discount rate if savings on future maintenance dredging at the Patawalonga are taken into account; and use of groynes became advantageous at a 2% discount rate. The saving decreases at higher discount rates. A risk factor of approximately $3m (PDV 5%) could apply because of design uncertainties. As for a major replenishment alone, a risk factor of up to $3m (PDV 5%) may apply for use of the Outer Harbor sand.
6. The 'offshore breakwaters with replenishment' alternative, at $26.9m or $29.4m (PDV 5%, and depending on the sand source), is the most costly protection method considered. Additional cost risk factors need to be taken into account, as for the 'groynes' alternative.
7. Present day values of beach loss are sufficient to conclusively exclude the 'no protection' and 'seawalls only' options from the financial comparison. The extra beach gain for a major beach replenishment over continuing the present measures has been estimated (Alternatives Study) at $5.6m (PDV 5%) to $8.6m (PDV 2%). However, the validity of applying the full value of this to the cost comparison is doubtful. Nevertheless, a major replenishment could have a real present day value advantage at low discount rates.

5.10.3 Capital Expenditure Implications

Continuation of the present measures is the only viable method that does not involve large capital expenditures in the first few years. The assumed and calculated cash flows for each alternative are shown in the cost tables in Appendix A.

It is evident from these that the largest annual expenditures will be for the replenishment component of all the methods that include a major beach replenishment, ie all viable strategies other than the present one. Costs of between $5m and $10m/year apply for the first 3 or 4 years, depending on the alternative. These sums would mainly be spent on a major dredging contract. As discussed in sections 5.2.2 and 5.7.5, and as assumed in the Alternatives Study, the initial dredging could be spread over 10 years, though this would still involve an initial cost of approximately $5m on having a dredge built for the purpose. Although slightly less costly on a present day value basis, this alternative has been rejected for other reasons.

Methods that have large cash flows would obviously need special budgeting at a state level and would affect other State Government projects. In commenting on the Alternatives Study, Treasury has advised that alternative strategies should be assessed on their comparative costs, with appropriate application of discount rates, and that the most economical method would receive favourable consideration for funds, without undue emphasis being placed on cash flow.

5.10.4 Cost Sharing with Local Government

No distinction has been made here between costs that would be borne separately by State or Local Government, and recommendations on this are beyond the scope of this report. However, this will need to be addressed in due course, and the following background information on past and present coast protection funding may be relevant.

The Coast Protection Act enables the Board to provide grants to councils for up to 80% of the cost of coast protection works and up to 100% for storm damage repairs. It also enables the Board to carry out works itself, and to recoup up to 20% of the cost from councils. However, this has never been necessary. Policy before the 1981 storms was that the Board paid two-thirds of the cost of seawalls and the full cost of beach replenishment. Dune restoration and maintenance has been carried out by councils with the Board contributing 80% of the cost. Councils have generally kept boat ramps and drains clear at their own cost.

Until 1981, the Board paid the full cost of storm damage repairs, though it was sometimes difficult to distinguish between storm damage repairs and new work that was required earlier than planned because of a particular storm.

The 1981 storms brought about a reassessment of this policy. The present policy is that the Board continues to carry out beach replenishment and pay the full cost of this, and that 80% grants are provided to councils for seawalls and similar projects and for storm damage, without distinction being made between these. Dune restoration continues to receive grants at an 80% level. The present policy is that drainage works, including outlets, do not attract grant monies.

Budgeting and cost sharing obviously needs to be addressed when both State and Local Government have had an opportunity to consider the findings of this review.

5.10.5 Beach Amenity

Comparison of the alternative methods based on beach amenity is implicit in the discussion in sections 5.5.1 and 5.9.1, and in the discussion of each alternative (sections 5.7.1 to 5.7.8). Only a brief summary is needed here.

The beach replenishment options (ie all options except 'no protection' and 'seawalls only') would give improved beaches, with these being provided soonest by a major replenishment. The beaches at Glenelg and Brighton would become continuous at all tides, though this might only be achieved in 20 to 30 years time for 'continuation of the present measures'. The present day value of this beach improvement has been valued at $11m and $5.1m for 'major replenishment' and 'continuation of the present measures' respectively. The new dunes and wider beaches could make it more difficult to launch boats from beaches, despite measures that would be taken.

Use of groynes or offshore breakwaters with beach replenishment would provide a more varied but interrupted beach. Groynes could detract from the coast's appearance, especially if additional groynes were found to be needed between those assumed here. They would provide fishing platforms and windbreaks, but would be likely to introduce swimming hazards. Offshore breakwaters would have a lesser effect, though might still cause loss of some beach between the structures.

Both the 'no protection' and 'seawalls without replenishment' options would result in beach losses at Seacliff, Brighton and Glenelg. These could be expected to be severe in 20 years time and almost total by the end of the 50-year comparison period. It is probable that only isolated small beaches would remain, at the Patawalonga groyne and possibly at one or two other places. The estimated present day values of this beach loss (at 5% discount rate) are $23.3m and $27.7m for 'no protection' and 'seawalls without replenishment' respectively.

A major replenishment would have a significant temporary effect on beach amenity for the first few years while sand was being pumped onto the beaches and distributed with earthmoving machinery. The present impacts of sand carting would persist for a continuation of the present measures. Trucks and loaders would continue to operate on beaches for part of each year, and there would continue to be temporary beach alteration due to removal and dumping of sand. A major replenishment would provide a respite from this for 10 or 20 years, after which it would need to be resumed, at a lower level, for top-up replenishment.

A major beach replenishment without groynes or breakwaters would clearly provide the best beach amenity. Continuation of the present measures would provide a satisfactory and progressively improving level of amenity. The effect on amenity of groynes or breakwaters remains debatable, though comment to date has been negative. The 'no protection' and 'seawalls without replenishment' alternatives would both result in serious loss of beaches with a consequent large reduction in beach amenity.

5.10.6 Comparison of Other Social and Environmental Aspects

A comparison of the alternatives on social and environmental grounds is implicit in Section 5.9, and particularly in its conclusions, Section 5.9.5. The following is a summary of the most salient points.

A major replenishment would involve the least environmental impact, provided that the offshore sand source is not located in seagrass beds, and provided that sand is pumped direct to shore from the hopper dredge or barges, and is not dumped on the seagrasses and rehandled. Trucking

for later top-up replenishment would be a significant impact, though much less than for continuation of the present measures. There is a possibility that rapid beach readjustment following the major replenishment could have a detrimental effect on nearshore seagrasses.

Despite having by far the greatest amount of truck movements, continuation of the present measures probably involves the next least overall environmental impact, though this depends on the weight attached to the trucking nuisance. A significant part of Torrens Island would have to be mined for replenishment sand.

The use of groynes or offshore breakwaters with replenishment would alter the beach's appearance and general amenity (the effect of this is compared in the previous section), and would have some impacts on seagrasses in the vicinity of the structures to be built south of Glenelg. Those to the north would be shoreward of the seagrass beds. Although the consultants for the Alternatives Study considered that the impacts would be small and not very significant in comparison with the seagrass recession presently occurring, there is a possibility, albeit slight, that an initial disturbance could initiate a more widespread loss. The use of groynes or breakwaters would cause eventual loss of the Semaphore beach and adjacent beaches.

The environmental consequences of 'no protection' or 'seawalls without replenishment' have not been considered, other than in regard to the beach loss discussed and compared in the previous section.

5.10.7 Effect of More Rapid Sea Level Rise

The comparison here has so far been based on a 2 mm/year sea level rise, which has been estimated to add 5,000 m^3/year to beach replenishment needs. The likelihood of more rapid sea level rise, due to the 'greenhouse' effect, is discussed in Section 2.3.6, having regard to information that became available towards the end of this review. Since higher rates of sea level rise could occur, the effect on the comparison of alternative strategies needs to be considered. Inclusion of this future aspect at this stage of the review is, however, necessarily general and not related to actual design or costing.

A 10 mm/year rise is assumed here for the 50-year period (see Section 2.3.6). The increase would be more likely to be gradual, with a lower rate of rise in earlier years and being higher towards the end of the period. This means that the conclusions of this review would remain valid for the immediate future at least.

The main effects of a more rapid sea level rise would be that loss of beaches and higher storm tide levels would occur sooner. More sand would be needed for beach replenishment, and structures would need to be higher and capable of withstanding larger waves. The substantial flooding effects, especially in the northern part of the study area, are not addressed here.

A 10 mm/year sea level rise would initially deplete the southern beaches at a rate in excess of 25,000 m^3/year (an estimate based on the simplistic method described in Section 2.6.3), and would later deplete beaches to the north, causing an exponential increase in the quantity of sand needed for beach replenishment.

Supplies of sand for continuing the present measures (with the appropriate increase in sand quantities) could be exhausted in 20 to 30 years, after which another strategy would need to be adopted. Nevertheless, economical protection could continue to be provided in the interim, deferring higher costs of alternatives and thus reducing the overall present day value.

A major replenishment, combined with increased top-up from onshore sources, would enable the beaches to be retained for longer, though it might need to be repeated during the 50-year period. The supporting top-up replenishment would need to be at approximately the same level as the present program. Because a large part of these costs would be in the more distant future, the effect on present cost value would not be extreme – probably in the range $5m to $10m depending on when the major replenishment needed to be repeated. This emphasises the importance of searching for offshore sand, an even larger quantity of which would be needed. Retention of the beaches would probably not be practical beyond 50 years.

Groynes and offshore breakwaters would become less attractive because they would cost more to build and because their benefits would reduce. The alongshore sand transport they interrupt would become a lesser proportion of the total sand loss, and the need for top-up would continue at a high level, with significant trucking still being needed. The performance of a strategy including such structures would become even more uncertain.

Seawalls could become the only viable strategy for the 50- to 100-year period, depending on the rate of sea level rise. Seawalls would need to be founded lower, be higher, and be constructed using larger armour stone. They would cost considerably more than those to the present design.

5.11 CONCLUSIONS

The following conclusions are drawn, taking into account the probable effectiveness of each method, the costs, and the environmental and social consequences. A 2 mm/year relative sea level rise is assumed, except in conclusion 11, which considers the effect of a higher rate of rise.

1. Coast protection costs of the order of those considered here are justified by the likely consequences of taking no action.
2. Constructing seawalls and not doing any further beach replenishment could be the least costly way of protecting property over the comparison period, depending on how long the present seawalls could survive the loss of beaches. However, the beach loss and the monetary value estimated for this would be very high compared to the savings over other strategies. This option is not considered to deserve further consideration.
3. Continuing the present coast protection measures has the lowest present day cost value and, depending on the success of the present offshore sand survey, may be the only viable strategy. At $13.4m (PDV 5%), it would be approximately $6m to $8m less than a major beach replenishment, which is the next least costly method. At low discount rates, cost differences between the strategies decrease, and present day values for the costs of continuing the present measures, a major replenishment or a major replenishment with groynes become approximately equal at rates of 2% to 3%. The cost advantage of the present policies increases with increasing discount rate and, at 7.5%, its present day value is approximately half that of a major beach replenishment.

   Although it has little impact on the physical or biological environment, continuing the present measures would involve nearly three times as much truck travel as the other alternatives considered, and would have by far the greatest impact on coastal residents.

   Onshore sand supplies are sufficient for the 50-year comparison period. It is assumed that the sand on Torrens Islands would be used, and that other more distant sources will also be available for beach replenishment.
4. A reduced, ‘status quo’, version of continuing the present measures offers little advantage either in cost saving or reduced trucking. It would not enable the present problems to be rectified, nor would it allow for any error in the estimates of sand loss. It is not considered a worthwhile strategy, though remains an option that could be reconsidered if beaches and dunes are built up more quickly than anticipated.
5. Major beach replenishment is a viable strategy if a suitable offshore sand source can be found. It is worth considering, despite the estimated present day cost values of $19.1m to $21.6m (5% discount rate) being higher than for continuing the present measures. It would enable trucking to be avoided for 10 to 20 years and to be at a reduced level after that. It would also provide a higher standard of protection and better beaches sooner and, providing that a sand source could be found in an area without seagrasses, would involve the least environmental impact and public inconvenience.
6. The Outer Harbor sand deposit, which has only been partly proven, is only marginally suitable because it is too fine. Sand would be lost from the replenished beaches, and the cost of

replacing this, if more than assumed in the costing here, could have a present day value of up to $3m. This source should only be considered if no better source can be found and only if adequate volumes of coarser sand are found in the area adjacent to that previously surveyed. If this source were to be used, the least costly and environmentally preferred method would be to dredge sand with a cutter-suction dredge and pump it to the southern beaches via a pipeline, which would cross under the shipping channel and be above ground for the remainder of the route. Use of this sand would affect future port expansion plans, and would need to be agreed with DMH.

7. The most likely location of suitable offshore sand is off the Onkaparinga River or off the coast south of this. A trailing suction hopper dredge would be the most practical and economical equipment for use of sand in this area. The method would involve little environmental damage, provided there was little marine vegetation at the sand source and provided that the sand was pumped ashore from the main dredge and not dumped and rehandled over the seagrasses. The former, preferred, method has been shown likely to have the lower cost.

8. Groynes or offshore breakwaters could be viable with a major replenishment, but not on their own. They would reduce the need for top-up replenishment with its associated costs and trucking nuisance, though the strategy would cost more despite this saving. They would also reduce the cost of maintenance dredging of a channel at the Patawalonga, assuming that such a channel were to be made, and they would help avoid stormwater drainage problems, which may result from beach replenishment. However, their performance is uncertain, mainly because of the unusually large variation in wave energy direction, and they cannot be considered as a viable alternative without first carrying out a full-scale experiment. Their use could result in a lower overall level of protection than would be gained from replenishment alone, and an erosion risk could be introduced at some places where it does not present exist.

   The present day value (5% discount rate) of using groynes with replenishment is $2.8m more than for replenishment alone. The difference reduces to $1m if maintenance dredging at the Patawalonga is taken into account; and the margin reduces further with decreasing discount rate, with use of groynes becoming economical at rates of 2% to 3%. If the groynes do not work satisfactorily at the spacings assumed, the extra cost of additional groynes or seawalls, which might be needed, could be up to $3m (PDV 5%).

   Offshore breakwaters would cost more, and their performance is even less certain than that of groynes, though their effects might not be as harsh.

   A hybrid solution, which uses groynes and breakwaters to the best advantage, would also be more expensive than using groynes, and the risks would remain.

   Use of groynes or breakwaters would alter the appearance of the beach and its suitability for different types of beach recreation. They would also result in some seagrass damage at and near some of the structures, though this is unlikely to spread or to be significant in relation to seagrass recession presently occurring. Erosion would occur north of the northern-most structure, causing eventual loss of beaches and foreshore reserves at Semaphore.

9. The trial groyne suggested in the Alternatives Study in the context of a major replenishment is not recommended at this stage, because its main purpose would be to provide information for a groyne strategy, which has not been shown to be economical and which is considered to involve too much risk, even if it were to work as assumed. Other information that the experiment would provide is not considered to be worth the cost or the risk associated with such a structure.

   A groyne at The Broadway might be worthwhile if built after a major replenishment and in conjunction with dredging a boating channel at the Patawalonga. It should not be considered without a major replenishment. It would facilitate and reduce maintenance dredging costs, and would provide a considerable length of wider beach and improved protection south of the structure. These advantages would be offset by the additional top-up replenishment, which would then be needed at North Glenelg as well as at Brighton. The loss of a short length of marginal beach north of The Broadway would be more than offset by the better beach south of the structure.

10. Other coast protection methods are either unsuitable for the local conditions or are insufficiently developed. These include floating breakwaters, artificial seaweed and shaped offshore dredging (to modify wave refraction). The groyne and breakwater strategies are partly based on the concept of artificial headlands, which cannot be considered as a separate or different method.
11. A possible higher rate of sea level rise associated with a projected warming of the earth would not alter short-term action recommended in this review. It might result in a continuation of the present measures being a viable strategy for only another 20 to 30 years. Depending on the availability of offshore sand, beaches could be retained by a major replenishment, possibly repeated in 30 to 40 years time. In the 50- to 100-year period, seawalls could become the only viable option, and attempts to retain beaches may need to be abandoned at that stage. Groynes or offshore breakwaters become less predictable and their retarding of alongshore sand movement becomes a less significant benefit.

5.12 STRATEGY RECOMMENDATIONS

The review has given rise to the following recommendations:

1. Continuing the present coast protection measures remains the only viable strategy until an offshore sand source is found. It is also the least costly way to improve the beaches and the level of protection. It is therefore recommended that the annual beach replenishment be continued at the present level of approximately 100,000 m^3 of unbulked sand (approximately 135,000 m^3 in trucks).

   Seawall construction should be limited to replacement of existing seawalls, and should only be done where adequate protection cannot be provided by the beach replenishment program. Information that might lead to a review of this strategy is unlikely to become available in less than 2 years, and the strategy should be continued for at least this period.
2. A ‘status quo’ alternative, involving a reduced level of beach replenishment to match the estimated sand losses, would not remove some present erosion problems and might not provide an adequate level of protection. This alternative is not recommended.
3. The search for an offshore sand source should continue, and a major replenishment should be reconsidered if sufficient suitable sand is found.
4. Use of groynes or other devices to control alongshore sand movement is inappropriate in the Adelaide conditions, and would have damaging effects. Such structures are consequently not recommended. Groynes could be considered if used with a major beach replenishment but would still introduce considerable risk, would offer few advantages, and would result in a higher cost.

Other recommendations are as follows:

5. The large sand deposit on the Electricity Trust’s property at Torrens Island should be reserved for future beach replenishment. Other, more distant, sand sources should be further investigated and suitable sand reserved.
6. The Outer Harbor sand source should not be considered for a major beach replenishment unless no better source can be found and unless further investigations at Outer Harbor show adequate volumes of coarser sand in the area adjacent to that previously surveyed. The use of this deposit would need to be carefully balanced against DMH long-term plans for reclaiming the area over it.
7. The Torrens Outlet should be cut through and bypassed regularly to avoid sand build-up and the consequent loss of sand from the Henley and Grange beaches.

8. A single longer groyne at The Broadway, South Glenelg, should be reconsidered in the future, if a major replenishment were to be done and if a channel were to be dredged at the Patawalonga.
9. If or when a rapid rate of increase in sea level is confirmed, increased sand carting, major replenishment, or seawalls should be considered, having regard to the rates of change and the results of the present offshore sand survey.
10. The proclaimed Management Plan for the Metropolitan Coast Protection District should be amended to take account of those recommendations accepted by the Board and recommended to the Minister.

Recommendations on future studies and investigations are provided in the next chapter.

CHAPTER 6: FUTURE STUDIES AND INVESTIGATIONS

This review has shown how dependent the coast protection strategy is on coastal processes, some of which are not well enough understood and which cannot yet be reliably quantified, and also on sand supplies that have yet to be found. This chapter reviews the need for future studies and survey work and indicates their priority. They are dealt with in the order of approximate priority, insofar as it is possible to distinguish this. The recommendations here relate to discussion elsewhere in the report, mainly in Chapter 2. This is referenced by section number, and additional explanation is kept to a minimum.

Given the Coast Protection Board's function and its limited staff and financial resources, emphasis must be placed on the most urgent studies, and on those likely to provide important and relevant information. However desirable or long-sighted it might be for the Board to support university studies of a related but more basic research type, present circumstances prevent this. However, a balance needs to be drawn between carrying out only applied research into immediate problems and collecting data or initiating studies of a longer-term type. It would be irresponsible to ignore the needs of the next generation whose coastal erosion problems seem likely to far exceed the present ones.

6.1 RELATIVE CHANGE IN MEAN SEA LEVEL

Refer to sections 2.3.2 to 2.3.6, and 5.10.7.

Recommendations

1. The Outer Harbor and Port Adelaide tide gauges should be maintained (or replaced if necessary) to ensure a high level of accuracy. The records from these and other gauges in South Australia and in the other states should be analysed to maintain the best possible, up-to-date information on rates of change of mean sea level.
2. The 5-yearly survey program to determine rates of coastal land settlement should be continued.
3. A new tide gauge should be established at Port Stanvac or Brighton, to obtain a better spread of information and to minimise the complications of land settlement.
4. A study should be done into the broader effects of sea level rise, to include flooding at Adelaide and effects on other parts of the state's coast. This should perhaps be combined with a statewide study into all likely effects of climatic change including those on agriculture and water supply.

6.2 THE SEDIMENT BUDGET AND ALONGSHORE TRANSPORT

Refer to sections 2.5 and 2.6, and references throughout Chapter 5.

Long-term surveys of sand volumes in the active beach zone, although costly, are likely to provide the most useful and reliable information on alongshore transport losses and on response to sea level change. Shorter-term surveys and detailed study is less likely to be profitable, because of the large seasonal and other short-term effects. However, the value of the long-term survey information can be enhanced through better understanding of the processes and of local effects, such as at structures.

Recommendations

1. The beach profiling program should be continued, with regular biannual measurement of most of the profiles and with 5-yearly surveys to a finer grid at North Haven, between the Patawalonga and Torrens Outlets, and at the main Brighton replenishment area.

2. The sediment transport model should be further developed and run over the survey period (1975 to present) to aid interpretation of the survey information.
3. The beach profiling should continue to be combined with underwater observation of sand movements against reference stakes. It is important to determine whether or not significant changes are occurring in deeper, nearshore waters and, if so, to improve the accuracy of the profile measurement in these deeper waters.
4. The seaward limit of significant sediment interchange should be determined. Tracer and sedimentological studies should be considered in conjunction with the study of nearshore seagrasses.
5. Sand movements at the Patawalonga groyne and channel should be further investigated with a view to determining the amount of natural sand bypassing, the amount lost offshore, and to refine quantity estimates for maintaining a boating channel.
6. The main sand source beaches at Point Malcolm and Semaphore and at the Torrens Outlet should be monitored to determine rates of sand depletion or accumulation and the response of the nearshore profile.

6.3 SAND SOURCES

Refer to sections 2.2.4, 5.2 and 5.4, and references throughout Chapter 5.

Recommendations

1. The recently initiated offshore sand survey should be extended if necessary to provide a thorough and complete assessment of offshore sand for beach replenishment, including proving of quantities. The survey should include further proving of the previously surveyed deposit at Outer Harbor.
2. If a new offshore deposit is found, an environmental study should be done to determine likely impacts and possible constraints. The possibility of altered wave refraction causing nearby wave concentration and coastal erosion would need to be investigated.
3. Further surveying should be carried out to locate new onshore deposits of sand suitable for beach replenishment.
4. A special study is needed into methods of working the Torrens Island sand deposit and rehabilitating the area afterwards. A trial excavation and restoration should be conducted as soon as practical to test the viability of creating an artificial samphire environment.

6.4 SEAGRASSES

Refer to Section 2.6.

A combined Department of Fisheries, University of Adelaide (Marine Biology) and consultant study has recently been approved for Australian Government research funding, and will be starting in early 1984. It will include studies of the species and density distribution between Glenelg and Henley; of the dynamics of the blow-outs and the sediment movements involved; of the plant biology (growth, life cycle, effect of epiphytic growth, etc); of smothering by sediment; and of transplantation techniques.

The Engineering and Water Supply Department will soon be completing a study into the effects of sewage treatment discharges. This will provide additional information.

Recommendations

1. The aforementioned combined seagrass study should be followed closely, with a view to determining any effects on the sediment budget when enough information becomes available.
2. The seagrass study should be supported by regular mapping of seagrass change, using aerial photography and possibly other remote sensing techniques.

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APPENDICES

APPENDIX A

PRESENT DAY VALUE COST CALCULATIONS

The assumptions and basis for the costing are set out in the main text, under each alternative heading.

The present day values are calculated according to the formula, applied to each year separately, and then summed:

$$PDV = \sum_{o}^{n} \frac{C}{(1 + \frac{r}{100})^{n-1}}$$

where PDV = present day value

C = expenditure at year n

n = number of years in the future

r = discount rate, as a percentage

Future expenditures are calculated at today's money values. The discount rate takes inflation into account.

ALTERNATIVE 2: CONTINUATION OF PRESENT MEASURES

Years	Sand $rn	Seawalls $rn	Torrens Outlet $rn	Dunes $rn	Mtnce $rn	Total $rn
1–10	.440	.150	.020	.010	.020	.640
11–15	.440	–	.020	.010	.050	.520
16–25	.625	–	.020	.010	.050	.705
26–50	.835	–	.020	.010	.050	.915

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	38.9	23.8	13.4	9.4	7.1
Costs For First 20 Years	12.5	10.4	8.2	6.9	5.9

ALTERNATIVE 2A: MAINTAINING THE STATUS QUO

Years	Sand $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1–10	.340	.150	.020	.020	.530
11–36	.340	.050	.020	.020	.430
37–50	.380	.050	.020	.020	.470

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	23.1	15.0	9.1	6.8	5.4
Costs For First 20 Years	9.6	8.1	6.4	5.5	4.7

ALTERNATIVE 3: SEAWALLS WITHOUT REPLENISHMENT

Years	Seawalls $m	Mtnce $m	Torrens Outlet $m	Total $m
1 & 2	.100	–	.020	.120
3	1.300	–	.020	1.320
4	1.100	–	.020	1.120
5	.200	.100	.020	.320
6	.900	–	.020	.920
7	.900	–	.020	.920
8	.700	–	.020	.720
9	.600	–	.020	.620
10	1.400	.100	.020	1.560
11–15	.800	.020	.020	.840
16–20	.800	.020	–	.82
21–25	.100	.020	–	.12
26–50	–	.020	–	.020

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	17.1	13.8	10.4	8.5	7.1
Costs For First 20 Years	16.0	13.2	10.1	8.3	7.0

TABLE B2 ESTIMATES OF SEAWALL REPLACEMENT REQUIRED FOR ALTERNATIVE 3

Section of Beach	Existing Protection	Length (m)	Length of Revetment Required (m)	When Likely to be Required
1. Burnham Road to Maitland Terrace	Dune	650	650	Year 3
2. Maitland Terrace to Bindarra Road	C	1,800	1,800	Years 3 and 4
3. Bindarra Road to Kent Street	A	2,000	–	–
	B	1,300	1,300	Years 1 to 13
	Dune	400	–	–
4. Kent Street to Patawalonga	D	850	–	–
5. Patawalonga to West Beach Road	B	2,500	2,500	Years 1 to 25
	Dune	850	850	Years 10 to 14
6. West Beach Road to Lexington Road	D	1,700	–	–
7. Lexington Road to Jetty Street (Grange)	D_2	1,350	1,350	Years 6 to 12
	C	1,275	1,275	Years 6 to 12
	A	300	–	–
8. Jetty Street to Third Street	Dune	425	425	Years 6 to 8
	Dune	4,750	4,750	Years 10 to 19
9. Third Street to Outer Harbor	D2	600	600	Year 20
	Dune	500	500	Year 20
	Dune	5,750	–	–
	D	400	–	–

From Kinhill Stearns and Riedel & Byrne (1983)

ALTERNATIVE 4: MAJOR REPLENISHMENT FROM OUTER HARBOR

Years	Seawalls $m	Major Replenishment $m	Top-up $m	Torrens Outlet $m	Mtnce $m	Total $m
1	–	4.750	–	.020	.020	4.790
2	.350	8.80	–	.020	.020	9.190
3	.350	4.35	–	.020	.020	4.740
4	–	1.26	–	.020	.050	1.330
5–10	–	–	–	.020	.050	.070
11–50	–	–	.160	–	.050	.210

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	28.9	24.8	21.6	20.2	19.2
Costs For First 20 Years	22.6	21.6	20.3	19.6	18.8

ALTERNATIVE 4A: MAJOR REPLENISHMENT FROM A MORE SUITABLE OFFSHORE SOURCE

Years	Seawalls $m	Major Replenishment $m	Top-up $m	Torrens Outlet $m	Mtnce $m	Total $m
1	–	7.8	–	.020	.020	7.84
2	.350	6.6	–	.020	.020	6.99
3	.350	1.7	–	.020	.020	2.09
4	–	–	–	.020	.050	.070
5–10	–	–	–	.020	.050	.070
11–50	–	–	.160	–	.050	.210

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	25.8	21.9	19.1	17.9	17.1
Costs For First 20 Years	19.5	18.7	17.8	17.2	16.8

ALTERNATIVE 5: MAJOR REPLENISHMENT WITH GROYNES

Years	Groynes $m	Major Replenishment $m	Top–up $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1	.50	4.75	–	.10	.020	–	5.37
2	.50	8.80	–	.35	.020	–	9.67
3	–	4.35	–	.35	.020	–	4.72
4	.40	2.46	–	–	.020	–	2.88
5	–	–	–	–	.020	.10	.12
6	.50	–	–	–	.020	–	.52
7	–	–	–	–	.020	–	.02
8	.50	–	–	–	.020	–	.52
9	–	–	–	–	.020	–	.02
10	.50	–	–	–	–	.10	.60
11–20	.25	–	–	–	–	.03	.28
21–50	–	–	.04	–	–	.03	.07

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	29.3	26.8	24.4	23.1	22.0
Costs For First 20 Years	27.2	25.8	24.0	22.9	21.9

ALTERNATIVE 5A: MAJOR REPLENISHMENT WITH GROYNES (NEW OFFSHORE SOURCE)

Years	Groynes $m	Major Replenishment $m	Top–up $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1	.50	7.8	–	.10	.020	–	8.42
2	.50	6.6	–	.35	.020	–	7.47
3	–	2.9	–	.35	.020	–	3.27
4	.40	–	–	–	.020	–	0.42
Years 5 to 50 as for Alternative 5							

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	26.3	24.0	22.0	20.9	20.0
Costs For First 20 Years	24.2	23.0	21.5	20.6	19.9

ADDITIONAL GROYNES/SEAWALLS FOR 5 AND 5A

Additional groynes or seawalls, which could be needed if the strategy does not work as well as envisaged in the Alternatives Study – refer to 'Disadvantages' heading in 5.7.5.

(a) Additional Groynes

All 11 groynes, as proposed, to be built in the first 10 years rather than in the first 20. Ten additional groynes to be built between them during years 11–20.

Year	Cost $m
1	–
2	–
3	.5
4	–
5	.5
6	–
7	.4
8	–
9	.5
10	.5
11–20	.25
21–50	–

Present Day Values $m				
0%	2%	5%	7.5%	10%
4.9	4.0	3.1	2.5	2.1

(b) Additional Seawalls

5 km of new seawall in Henley–West Lakes area at $1,110/m (to be consistent with Alternatives Study). Half to be built between year 10 and 20 and half between year 20 and 30.

2 km of seawall upgrading at North Glenelg at years 9 and 10, to follow 2 years after construction of groyne no.4 at West Beach (when it may be discovered that this groyne is inadequate). AE $600/m.

Year	Cost $m
1–8	–
9–10	.5
11–30	.28
31–50	–

Present Day Values $m				
0%	2%	5%	7.5%	10%
6.8	4.8	3.0	2.1	1.5

ALTERNATIVE 6: MAJOR REPLENISHMENT WITH OFFSHORE BREAKWATERS

Years	Groynes $m	Major Replenishment $m	Top–up $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1	.80	4.75	–	.10	.020	–	5.67
2	.80	8.80	–	.35	.020	–	9.97
3	–	4.35	–	.35	.020	–	4.72
4	.80	2.46	–	–	.020	–	3.28
5	–	–	–	–	.020	.10	.12
6	.80	–	–	–	.020	–	.82
7	–	–	–	–	.020	–	.02
8	1.0	–	–	–	.020	–	1.02
9	–	–	–	–	.020	–	.02
10	1.0	–	–	–	–	.10	1.10
11–20	.52	–	–	–	–	.03	.55
21–50	–	–	.04	–	–	.03	.07

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	34.3	31.0	27.6	25.7	24.2
Costs For First 20 Years	32.2	29.9	27.2	25.5	24.1

ALTERNATIVE 6A: MAJOR REPLENISHMENT WITH OFFSHORE BREAKWATERS (NEW OFFSHORE SAND SOURCE)

Years	Groynes $m	Major Replenishment $m	Top–up $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1	.80	7.8	–	.10	.020	–	8.72
2	.80	6.6	–	.35	.020	–	7.77
3	–	2.9	–	.35	.020	–	3.27
4	.80	–	–	–	.020	–	0.82
Year 5 to 50 as for Alternative 6							

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	31.3	28.2	25.1	23.5	22.2
Costs For First 20 Years	29.2	27.1	24.7	23.3	22.1

ALTERNATIVE 7: HYBRID SOLUTION

Years	Groynes/ Breakwaters $m		Major Replenishment $m	Top–up $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1	.50	–	4.75	–	.10	.020	–	5.37
2	.50	–	8.80	–	.35	.020	–	9.67
3	–	–	4.35	–	.35	.020	–	4.72
4	–	.80	2.46	–	–	.020	–	3.28
5	–	–	–	–	–	.020	.10	.12
6	.50	–	–	–	–	.020	–	.52
7	–	–	–	–	–	.020	–	.02
8	.50	–	–	–	–	.020	–	.52
9	–	–	–	–	–	.020	–	.02
10	–	1.0	–	–	–	–	.10	1.10
11–20	–	.62	–	–	–	–	.03	.65
21–50	–	–	–	.04	–	–	.03	.07

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	33.9	30.4	27.0	25.0	23.5
Costs For First 20 Years	31.8	29.4	26.5	24.8	23.4

ALTERNATIVE 7A: HYBRID SOLUTION (NEW OFFSHORE SAND SOURCE)

Years	Groynes/ Breakwaters $m		Major Replenishment $m	Top–up $m	Seawalls $m	Torrens Outlet $m	Mtnce $m	Total $m
1	.50	–	7.8	–	.10	.020	–	8.42
2	.50	–	6.6	–	.35	.020	–	7.47
3	–	–	2.9	–	.35	.020	–	3.27
4	–	.80	–	–	–	.020	–	0.82
Year 5 to 50 as for Alternative 6								

	Present Day Values $m				
Discount Rate	**0%**	**2%**	**5%**	**7.5%**	**10%**
Costs over 50 Years	30.9	27.6	24.5	22.8	21.5
Costs For First 20 Years	28.8	26.5	24.1	22.5	21.4

PATAWALONGA DREDGING MAINTENANCE

Alternative 2: Continuation of Present Policies

Year	Cost $m
1	–
2–50	0.2

Present Day Values for 50 Yrs $m				
0%	2%	5%	7.5%	10%
3.7	2.6	1.7	1.4	1.1

Alternative 3: Seawalls without Replenishment

Year	Cost $m
1	–
2–5	0.2
6–10	0.1
11–50	.06

Present Day Values $m				
0%	2%	5%	7.5%	10%
3.7	2.6	1.7	1.4	1.1

Alternative 4: Major Replenishment

As for Alternative 2.

Alternatives 5,6 and 7: Major Replenishment with Groynes, Offshore Breakwaters or Both

Year	Cost $m
1	–
2–50	0.1

Present Day Values $m				
0%	2%	5%	7.5%	10%
4.9	3.1	1.8	1.3	1.0

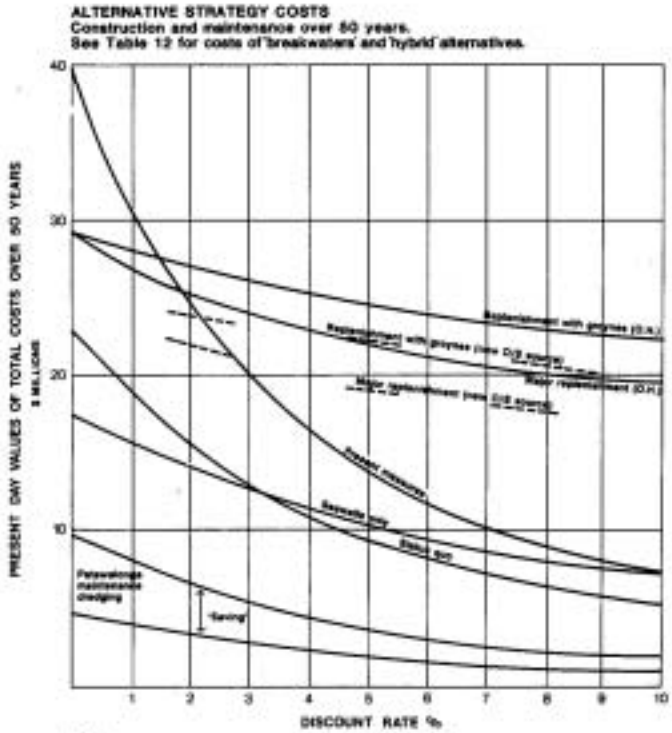
APPENDIX B

TABLE 12: CONSTRUCTION AND MAINTENANCE COSTS OF STRATEGIES

Present Day Values at 0%, 2%, 5%, 7.5% and 10% Discount Rates

No.	Alternative	Present Value of Costs, $ m						
		Costs Over 50 Yrs.					First 20 Yrs.	
		0%	2%	5%	7.5%	10%	0%	5%
1	No Protection	-	-	-	-	-	-	-
2	Continuation of Present Measures	38.9	25.8	13.4	9.4	7.1	12.5	8.2
2A	Maintaining the Status Quo	23.1	15.0	9.1	6.8	5.4	9.6	6.4
3	Seawalls Without Replenishment	17.1	12.8	10.4	8.5	7.1	14.8	10.1
4 (4A)	Major Beach Replenishment (U.H.) (From new offshore source)	26.9 (25.8)	24.8 (21.9)	21.4 (18.1)	18.2 (17.9)	15.7 (17.1)	22.6 (19.5)	20.3 (17.8)
5 (5A)	Major Replenishment with Groynes (U.H.) (From new offshore source)	29.3 (26.3)	26.8 (24.6)	24.4 (22.0)	23.1 (20.9)	22.0 (20.0)	27.2 (24.2)	24.0 (21.5)
6 (6A)	Major Replenishment with Offshore Breakwaters (U.H.) (Replenishment from new offshore source)	34.3 (31.3)	31.0 (28.3)	27.6 (25.1)	25.7 (23.5)	24.2 (22.2)	32.2 (29.2)	27.2 (24.7)
7 (7A)	Hybrid Solution (U.H.) (Replenishment from new offshore source)	33.9 (30.9)	30.4 (27.6)	27.0 (24.5)	25.0 (22.8)	23.5 (21.5)	31.8 (28.8)	26.9 (24.1)

"U.H." indicates use of the partly proven Outer Harbor sand source.
"New Offshore Source"--a suitable source yet to be found.



Note:
Financial risk factors are not included
- for replenishment from Outer Harbor up to $3.0 mill. additional.
- for groynes or breakwaters as above. $3.0 mill. and up to $3.0 mill. for additional structures.

TABLE 8: PRESENT DAY VALUE OF BEACH LOSSES AND GAINS

(Values in millions of dollars)

No.	Alternative	Discount Rate		
		2%	5%	10%
1	No Protection	(50.1)	(23.3)	(8.7)
2	Continuation of Present Measures	11.2	5.1	1.9
3	Seawalls Without Replenishment	(59.3)	(27.7)	(10.4)
4	Major Beach Replenishment	19.8	11.0	3.6
5	Major Replenishment with Groynes	19.8	11.0	3.6
6	Major Replenishment with Breakwaters	19.8	11.0	3.6
7	Hybrid Solution	19.8	11.0	3.6

Data from Kinhill Stearns and Riedel & Byrne (1983).

(...) indicates negative values i.e. beach loss.

TABLE 9: PATAWALONGA MAINTENANCE DREDGING COSTS

No.	Alternative	Present Value of Costs ($m) (discount rate)				
		0%	2%	5%	7.5%	10%
1,2A 4 4A	Continuation of Present Measures Maintaining Status Quo Major Replenishment	9.8	6.2	3.6	2.6	2.0
3	Seawalls without Replenishment	3.7	2.6	1.7	1.4	1.1
5 6 7	Replenishment with Groynes Replenishment with Breakwaters Hybrid Solution	4.9	3.1	1.8	1.3	1.0