EUROBODALLA SHIRE COUNCIL

BATEMANS BAY COASTLINE HAZARD MANAGEMENT PLAN

DRAFT FOR PUBLIC EXHIBITION

NOVEMBER 2001

WEBB, McKEOWN & ASSOCIATES PTY LTD

EUROBODALLA SHIRE COUNCIL

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This Coastline Hazard Management Plan for the Batemans Bay area was prepared by the Batemans Bay Coastal Management Committee for Eurobodalla Shire Council with funding assistance from the Department of Land and Water Conservation. The Plan is based on the assessments contained in the detailed planning report attached (or available from Council).

The Plan covers the foreshore and back beach areas of the Batemans Bay embayment downstream of the Princes Highway Bridge. The specific areas addressed include (see Figure i):

- Central Business District,
- Beach Road (Boat Harbour West),
- Wharf Road,
- Surfside Beach,
- Cullendulla Beach,
- Long Beach,
- Maloneys Beach,
- Hanging Rock (Boat Harbour East),
- Corrigans Beach,
- Caseys Beach.

The assessments are based on the procedures set out in the NSW Government's Coastline Management Manual (1990). The Plan concentrates on coastline hazard issues, but also considers wider coastal management issues such as:

- land ownership and tenure,
- aesthetics and ecological factors,
- recreational amenity,
- social issues,
- economic issues,
- climate change uncertainty.

Strategic objectives from the NSW Coastal Policy addressed by the Plan are:

- to manage the coastline and estuary environments in the public interest to ensure their health and vitality,
- to foster new initiatives and facilitate the continued involvement of the community in programs aimed at the restoration and rehabilitation of degraded coastal areas,
- to give the impacts of natural processes and hazards a high priority in the planning and management of coastal areas,
- to recognise and consider the potential effects of climate change in the planning and management of coastal development,
- to identify and protect areas of high natural or built aesthetic quality,
- ensure local government coastal policy and management is integrated and involves community participation and information exchange.

Preferred management options for each area were selected based on the assessments undertaken in the detailed planning report plus the outcomes of a series of community workshops and Council meetings. The attached Table i) summarises the preferred management options which are now provided for further public exhibition and comment prior to final adoption by Council.

Table i): Summary of Do Nothing and Preferred Management Options

Area	Management Option	Description/Comment	50 Year PV Cost ($'000)	Current Program Costs ($'000)		
				Short	Medium	Long
CBD	Do Nothing	Present day damages over 50 years	1 500			
	Minimum Floor Levels/Building Protection					
	Training Wall Repair	Immediate 300 t rock plus reshaping		200		
Beach Road (Boat Harbour - West)	Do Nothing	Present day damages over 50 yrs (not including BH)	500			
	Minimum Floor Levels/Building Protection					
	Training Walls	4000 t rock topping and/or widening			150/350	>350
Wharf Road	Do Nothing	Present day damages over 50 years	850			
	Voluntary Purchase	Rationalise foreshore land ownership		unknown		
	Minimum Floor Levels *					
	Training Walls (western section)*	4000 t rock topping			500	
Surfside Beach	Do Nothing	Present day damages over 50 years	400			
	Minimum Floor Levels/Building Protection					
	Beach Nourishment	Initial contingency plus ongoing sand over 50 yrs		200	200	>350
Cullendulla Beach	Do Nothing	Assumed loss over 50 years	>600			
	Relocate Assets	Move rising main, telephone line			500	
Long Beach	Do Nothing	Present day damages over 50 years	300			
	Minimum Floor Levels /Building Protection					
	Building Setbacks *	As per revised hazard lines				
	Beach Nourishment (eastern section) *	Bay Rd only, ongoing sand over 50 yrs			150	>300
Maloneys Beach	Minor Actions	Flood Study & Beach Ramp Maintenance			40	
Hanging Rock (Boat Harbour - East)	Do Nothing	Present day damages over 50 years	500			
	Levees	Fill & drainage, local then wider protection			300	>450
Corrigans Beach	Do Nothing	Present day damages over 50 years	250			
	Voluntary Purchase	Caravan park				>500
	Minimum Floor Levels *					
	Planned Retreat *	Move first row during re-development				unknown
Caseys Beach	Do Nothing	Present day damages over 50 years	450			
	Building Protection*	400 m wave deflection wall & ongoing road repair		100	150	>450

* Partial Option which does not provide full hazard protection.
- 50 year Present Value costs based on 7% discount rate.
- Short Term = less than 5 years.
- Medium Term = 5 to 10 years.
- Long Term = greater than 10 years.

1. PLANNING REPORT SUMMARY

1.1 Background

Preparation of this Coastline Hazard Management Plan report was initiated by Eurobodalla Shire Council and follows on from the coastal hazards review undertaken for the *Batemans Bay Vulnerability Study* (CRMD, 1996).

The main objectives of the project were to review the work undertaken for the Vulnerability Study, analyse and incorporate additional data gathered since 1996, and determine preferred coastal hazard management options based on the new information. The new information included recent storm and wave data, and the findings of the *Batemans Bay/Clyde River Estuary Processes Study Draft* (WBM, 1999)

The assessments presented in this report are based on the procedures set out in the NSW Government's Coastline Management Manual (NSWG, 1990). In addition to coastline hazards, the assessments include consideration of wider coastal management issues such as :

- land ownership and tenure,
- aesthetics and ecological factors,
- recreational amenity,
- social issues,
- economic issues,
- climate change uncertainty.

Initiation of this study predates the announcement of the NSW Coastal Policy 1997. This Policy places emphasis on Policy Goals and Objectives, and Strategic Actions. To better reflect the requirements of the Coastal Policy, some changes have been incorporated into the planning report.

The Plan is essentially targeted towards sustainable coastline hazard management and addressing relevant issues. The findings presented in this report will be use by Eurobodalla Shire Council, the Batemans Bay Coastal Management Committee (CMC), and the local community, when addressing coastline hazard issues and formulating wider coastal management plans for Batemans Bay.

1.2 NSW Coastal Policy 1997

The NSW Coastal Policy 1997 has as its central focus the philosophy of ecologically sustainable development. The principles of ESD as set out in the Coastal Policy are:

- conservation of biological diversity and ecological integrity,
- inter-generational equity,
- improved valuation, pricing and incentive mechanisms,
- the precautionary principle.

Strategic Objectives from the Coastal Policy addressed by the Plan include:

- To manage the coast line and estuary environments in the public interest to ensure their health and vitality (Policy Objective 1.4 - Actions 1.4.3 and 1.4.5).

- To foster new initiatives and facilitate the continued involvement of the community in programs aimed at the restoration and rehabilitation of degraded coastal areas (Policy Objective 1.5 - Action 1.5.1).

- To give the impacts of natural processes and hazards a high priority in the planning and management of coastal areas (Policy Objective 2.1 - Action 2.1.1).

- To recognise and consider the potential effects of climate change in the planning and management of coastal development (Policy Objective 2.2 - Action 2.2.2).

- To identify and protect areas of high natural or built aesthetic quality (Policy Objective 3.1 - Action 3.1.2).

- To ensure local government coastal policy and management is integrated and involves community participation and information exchange (Policy Objective 9.3 - Action 9.3.2).

1.3 Study Area

The study area includes the foreshore and back beach areas of the Batemans Bay embayment (see Figure 1). Generally, rocky foreshores and the estuary area upstream of the Princes Highway Bridge are not significantly affected by coastal hazards and have not been considered. However, the bay beaches and the southern training wall have been included.

To facilitate consideration of the hazards, and preparation of the Management Plan, the study area was divided into the following geographic/social zones (see Figure 1):

Inner Zone
- Central Business District
- Beach Road (Boat Harbour West) Area
- Wharf Road Area

Northern Zone
- Surfside Beach
- Cullendulla Beach
- Long Beach
- Maloneys Beach

Southern Zone

- Hanging Rock (Boat Harbour East) Area
- Corrigans Beach
- Caseys Beach

1.4 Hazards Definition

The coastal hazards identified in the study area and addressed by the Vulnerability Study and this Coastline Hazard Management Plan are:

- *coastal inundation*, or the flooding of beach and back shore areas by elevated ocean levels which occur as a result of storm conditions (combined with astronomic tides). Elevated ocean levels are mainly caused by wind and wave action pushing water on to the coast and low pressure barometric effects.

- *shoreline recession*, which is the progressive loss of beach sands over time due to movement off the beach and out of the beach embayment. These sands are not replaced over time. Shoreline recession associated with climate change related sea level rise is usually considered as part of this hazard.

- *beach erosion*, or the loss of sand from the beach dune system during storms. This hazard is often called "storm bite" because it can vary along a beach depending on the formation of rip cells. Storm erosion is repaired by beach building processes during quiescent periods.

- *sediment accretion*, caused by wind or water movement, which can cover valuable habitat areas such as rocky reefs, damage infrastructure and/or block access ways.

- *slope stability*, caused by waves or currents attacking the face of a slope and causing it to become unstable. In Batemans Bay this mainly applies to dumped rock seawalls.

1.5 Hazards Review

For this study the coastal hazards identified in the Batemans Bay Vulnerability Study were reviewed and quantified for each study area. Where necessary additional investigations were undertaken. A description of this work is presented in Chapters 2 to 13, and a summary of the coastal hazards is given in Table 1.

The review includes coastal inundation based on estimated foreshore setup levels, climate change and wave overtopping effects. It also includes shoreline recession and accretion, beach erosion, and seawall revetment stability.

The major hazard identified for the inner bay areas (the CBD, Beach Road and Wharf Road), was ocean inundation as a result of high astronomic tides combined with storm surge (wind stress and barometric effects during major storms), some minor Clyde River flooding effects, plus wave setup, runup and overtopping of the foreshore. Local catchment runoff was not found to be a major part of the problem except along Hanging Rock Creek in the Beach Road area. In the Wharf Road area sediment movements were also found to be a potential problem.

The major hazards for the northern zone beaches were found to differ from beach to beach. Surfside Beach and Long Beach were found to be subject to coastal inundation and beach erosion during storms, with some ongoing recession potential associated with climate change sea level rise. Cullendulla Beach was found to have a long term recession problem, as well as being subject to severe coastal inundation and some storm erosion. Maloneys Beach was subject to storm bite, but not at levels likely to cause a significant hazard.

Wave overtopping inundation was identified as the main coastal hazard for the southern zone (although local catchment runoff is a major contributing factor in this area). The Hanging Rock Creek area east of the Boat Harbour was found to be subject to significant foreshore wall overtopping and damage. Corrigans Beach was subject to coastal inundation and beach erosion, and some climate change induced recession. Caseys Beach was found to be subject to coastal inundation and wave damage to the foreshore revetment wall, the roadway along the beach, and the back beach area.

Table 1: Summary of 1% AEP Coastal Hazards for Batemans Bay

Area		Average Dune/Wall Height (mAHD)	Assessed 1% AEP Foreshore Setup (mAHD)	Theoretical Nearshore Wave Height (m)	Theoretical Wave Runup (m)	Estimated Overtopping Rate (m^3/s)	Adopted Beach Erosion Rate ($m^3/m/event$)	Adopted Beach Recession Rate (m/50 years)	Adopted Foreshore Inundation Level (mAHD)	Adopted Backshore Inundation Level (mAHD)
CBD		1.7 to 2.2	2.0	1.0	1.1	360	-	-	2.4	2.2
Beach Road	**(BH West)**	1.8 to 2.2	2.0	1.5	1.6	450	-	-	2.4	2.2
Wharf Road	**West**	1.5 to 2.0	1.8	1.4	1.4	160	-	-	2.4	2.0
	East	1.5	1.8	1.4	1.8	160			2.5	2.0
Surfside Beach	**West**	2.5	2.8	1.5	1.4	100	25	5	<3.8	2.3
	East	3.0	2.8	1.5	1.0	-	40	5	<3.8	2.3
Cullendulla Beach	**West**	1.5 to 2.0	2.0	1.5	0.8	250	20	70	>2.2	2.2
	Middle	1.5 to 2.0	2.0	-	-	-	8	28	>2.2	2.2
	Creek	-	2.0	-	-	-	-	90	-	2.2
Long Beach	**West**	5.0	2.7	1.5	2.0	zero	20	8	-	-
	Middle	5.0	2.7	1.5	1.5	5	35	8	-	3.5
	East	3.0 to 3.5	2.7	1.5	1.2	15	10	8	>2.7	3.5
Maloneys Beach	**Middle**	6.0	2.9	1.6	2.1	-	9	12	<3.6	3.5
Hanging Rock	**(BH East)**	1.8 to 2.2	2.0	1.4	2.1	810	-	-	2.5	2.0
Corrigans Beach	**North**	3.5	2.5	1.5	0.9	zero	-	6	-	-
	South	2.5	2.5	1.5	0.9 to 4.0	400	40	6	2.6	2.5
Caseys Beach	**North**	4.0	2.6	2.5	4.6	-	-	-	<5.0	>2.5
	South	3.0	2.6	2.5	4.6	140	-	-	<5.0	>2.5

1.6 Management Options Assessment

Following on from the hazards review, possible options for management of the identified coastal hazards were assessed. This work provides necessary information for the formulation of a Coastal Management Plan by Council, the CMC and the community. A description of the management options is presented in Chapters 4 to 13, and summarised in the following Section 1.7.

A wide range of options was examined for their social, environmental and economic impacts including do nothing, environmental planning, development controls, dune management and protective works as set out on the NSW Government's Coastline Management Manual (NSWG, 1990).

Note, not all the management options identified in the Manual are relevant to the coastal hazards and environmental conditions identified for the study area. The options specifically addressed in this report are:

- Do Nothing,
- Environmental Planning:
    - buffer zones,
    - restrictive zonings,
    - planned retreat,
    - voluntary purchase,
- Development Controls:
    - building setbacks,
    - minimum floor levels,
    - building protections (raising floors and hazard proofing),
    - relocatable assets,
- Dune Management,
- Protective Works:
    - dune/beach nourishment,
    - wave barriers,
    - back beach drainage,
    - training walls,
    - seawalls,
    - groynes,
    - offshore breakwaters.

The present value of the likely inundation damages used in the assessment was based on a standard Average Annual Damage (AAD) approach. In this method the average annual damage was estimated by multiplying the damage that can occur for a particular event by the probability of that event occurring in a given year. The numbers were then summed across the range of events. The present value of the damages was calculated assuming a 7% discount rate over a 50 year planning period.

1.7 Options Summaries

1.7.1 Central Business District

Details of the CBD assessment are given in Chapter 4. The main coastal hazards affecting the CBD are inundation as a result of wave overtopping of the training wall and wave impacts along the immediate foreshore during major storm/tide events with a frequency around once in twenty years. The assessment of management options found (see also Table 2 and Figures 2 and 2a):

- **Do Nothing** - the present value (PV) of the likely damages would be around $1.5 million over a 50 year planning period but the social dislocation likely to be caused would make the real costs much higher.

- **Environmental Planning** - options such as voluntary purchase or planned retreat are not feasible because of the extent and value of the existing development and the disruption they would cause to ownership and management of the CBD.

- **Development Controls** - existing minimum floor level freeboards are probably adequate for residential developments (0.65 m) and commercial properties (0.4 m) but are low in the immediate foreshore area (0.2 m) although development in this area is also required to consider wave impacts. The cost of building protection is around $1.4 million. Both the minimum floor level and the building protection options have moderate to low impact on the amenity of the area and its use.

- **Protective Works** - reshaping and additional rock are required to repair the existing training wall at a cost of around $200 000. Construction of a concrete wave barrier/levee along the foreshore with associated road and drainage works would cost around $1.7 million. Alternatively, raising and widening the rock wall would cost in excess of $2.0 million. Both these options would impact on the visual amenity of the foreshore area.

Based on the above, continuation of the existing development controls, possibly with some modifications/tightening for foreshore development, would appear to be the best coastline hazard management option. More detailed consideration of the foreshore wave barrier option could also be considered.

Table 2: CBD Management Option Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	1 500	-	-	-	VH	H	H
Environmental Planning:								
- restrictive zoning	not feasible	>5 000	VH	M	-	VH	VH	-
- voluntary purchase	not feasible	>5 000	H	-	-	VH	VH	-
Development Controls:								
- minimum floor levels	new development only	-	-	-	-	M	M	M
- building protection	modify 37 commercial, raise 6 residences	1 400	-	L	-	L	H	M
Protective Works:								
- training wall (repair)	immediate 3000 t rock plus reshaping	200	-	-	-	-	L	L
- wave barrier/levee	800 m wave deflection wall and drainage	1 700	-	H	L	L	H	-
- training wall (raising)	11000 t rock and fill plus drainage	2 000	L	VH	L	L	H	-

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.2 Beach Road

Details of the Beach Road area assessment are given in Chapter 5. The main coastal hazard in the Beach Road area is coastal inundation as a result of waves overtopping the training wall during major infrequent storm/tide events. There could also be significant wave impact damage to boats and infrastructure around the boat harbour, and combined catchment flooding/coastal inundation problems along Hanging Rock Creek. The assessment of management options found (see also Table 3 and Figures 3 and 3a):

- **Do Nothing** - the present value (PV) of the likely inundation damages would be around $500 000 over a 50 year planning period, although this would be much higher if there was extensive damage in the boat harbour area (this requires more investigation).

- **Environmental Planning** - there is already a large buffer zone and options such as voluntary purchase are not feasible because of the extent and value of the existing development.

- **Development Controls** - existing minimum floor level freeboards are probably adequate for coastal inundation (0.6 m foreshore, 0.8 m backshore). The cost of implementing a program of hazard proofing such as house raising would exceed $400 000, not including the boat harbour area.

- **Protective Works** - raising the existing training wall to prevent substantial wave overtopping would cost around $150 000 or $350 000 if widening of the wall was required. The visual and recreational impact would be limited because of the existing wall and park location. To raise the entire wall could cost up to $700 000.

Based on the above, continuing the existing minimum floor level policy would be recommended as would further investigations at a local level to determine the likely impact of wave overtopping on the marina/boat harbour facilities and vessels. Raising the training wall would also appear to be a feasible option.

Table 3: Beach Road Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	500	-	-	-	H	M	M
Environmental Planning: - voluntary purchase	 not feasible	 -	 M	 -	 -	 H	 VH	 -
Development Controls: - minimum floor levels - building protection	 new development only partial solution - raise 10 residences	 - 400	 - -	 - -	 - -	 L L	 M M	 M M
Protective Works: - training wall (raising)	 4000 t rock topping/6000 t widening	 500/700	 L	 L	 -	 -	 H	 -

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.3 Wharf Road

Details of the Wharf Road area assessment are given in Chapter 6. The main coastal hazards for both the western rock wall and the eastern beach sections of the Wharf Road area are coastal inundation and wave impacts as a result of wave overtopping of the foreshore during major infrequent storm/tide events. Sediment movement associated with a long term cycle of flood/non-flood foreshore erosion and accretion is also a problem. The assessment of management options found (see also Table 4 and Figures 4 and 4a):

- **Do Nothing** - the present value (PV) of the likely inundation damages would be around $850 000 over a 50 year planning period. No value was assigned to sediment movement damages.

- **Environmental Planning** - buffer zones and restrictive zonings were not considered to be effective methods of minimising damage. Voluntary purchase may be feasible for both the western caravan park development and for the eastern beach area. Some land ownership rationalisation may be desirable along the eastern foreshore.

- **Development Controls** - the existing minimum floor level freeboard (0.8 m) is adequate to prevent inundation and should help minimise any wave runup impacts. Controls on permanent caravan park developments near the foreshore should also assist. The cost of implementing a program of building protection, such as house raising and land raising, was estimated to exceed $2.5 million.

- **Protective Works** - raising the existing foreshore training wall to protect development was estimated at around $500 000, but may have negative amenity impacts on the caravan park. The provision of a rock wall or sand dune to also protect the eastern foreshore area was estimated at $1.4 and $1.1 million respectively. Depending on the alignment, a fixed wall option could result in wider sediment movement problems.

Based on the above, continuing with the existing floor level development controls would be advisable, but the associated building protection, particularly large scale landfills, is unlikely to be feasible. Upgrading and raising the rock wall along the western section of Wharf Road would appear to be an economically feasible option, but with some negative social/user impacts. Voluntary purchase may be suitable for both sections, possibly with some land ownership rationalisation along the eastern foreshore.

Table 4: Wharf Road Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	850	VH	-	L	H	H	H
Environmental Planning:								
- buffer zones	not feasible	-	VH	L	H	VH	H	M
- restrictive zonings	not feasible	-	VH	L	M	VH	H	M
- voluntary purchase	feasible	unknown	M	L	L	M	M	-
Development Controls:								
- building setbacks	justifiable	-	VH	L	L	VH	H	M
- minimum floor levels	partial solution only	-	-	-	-	M	H	M
- building protection	fill land & raise development	>2500	-	H	-	H	VH	M
- relocatable assets	not feasible	-	-	-	-	VH	M	M
Protective Works:								
- training wall	western 4000 t/ east 1000 t rock	500/1400	M	VH	VH	VH	H	-
- beach nourishment	400 t rock, 16000 m^3 sand	1100	M	H	H	H	H	-

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.4 Surfside Beach

Details of the Surfside Beach assessments are given in Chapter 7. The coastal hazards for the Surfside Beach area are coastal inundation, mainly wave runup and overtopping between properties on the beach dune, and a combination of coastline recession as a result of climate change and storm bite erosion. Storm bite erosion has not historically been a hazard but appears to have developed over recent years as a result of the depleted condition of the nearshore zone as part of the sand movement flood/non-flood cycle. The assessment of management options found (see also Summary Table 5 and Figures 5 and 5a):

- **Do Nothing** - the present value of coastal inundation and erosion damages would be around $400 000 over a fifty year planning period.

- **Environmental Planning** - options such as voluntary purchase are not feasible because of the extent and value of the existing development.

- **Development Controls** - existing minimum floor level freeboards are probably adequate (0.5 m) for back beach residential developments, but are not adequate to prevent wave runup damage along the foreshore dune. The cost of implementing a program of works in the short term to hazard proof existing development would be around $550 000, but could be significantly less (around $300 000) if erosion protection works are delayed until the problems become more immediate.

- **Dune Management/Protective Works** - beach reshaping and nourishment to raise the dune to prevent overtopping during major storms would cost around $200 000. This could include contingency work on temporary storm erosion protection. Substantial nourishment as a buffer against future storm erosion would cost around $550 000. Staged nourishment when necessary (erosion scarp within 15 m of a residential building) would cost around $200 000 every 10 to 15 years (total >$750 000 over 50 years) with a present value cost around $400 000.

- **Protective Works** - A seawall would provide a more permanent solution but would be more expensive (around $1 800 000 for a full beach wall), and would result in the sandy beach being lost. This would be both socially and economically undesirable. A shorter wall would be proportionally less (around $450 000 for a 200 m wall). Constructing back beach drainage would be very costly and would not fully address the hazards. Constructing groynes or a breakwater to diffract wave energy away from the beach would cost at least $1 500 000 and could adversely impact on other areas.

Based on the above, nourishing the beach would be economically feasible (and would probably be socially feasible) if minimal work was undertaken initially for overtopping plus some contingency erosion protection, and major work was delayed until necessary. A seawall resulting in loss of the sandy beach amenity is unlikely to be socially or economically feasible. Continuing the current policy of minimum floor levels with additional hazard proofing requirements in the immediate foreshore area would be economically feasible, particularly if erosion protection works were delayed until the problems became more immediate.

Table 5: Surfside Beach Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	400	L	-	L	VH	H	H
Environmental Planning:								
- voluntary purchase	not feasible	>5000	H	H	L	VH	VH	-
Development Controls:								
- minimum floor levels	partial solution only	-	-	-	-	L	L	M
- building protection	modify 20 residences and protect 3	550 (PV 300)	-	L	-	L	M	M
Protective Works:								
- beach nourishment	initial 6000 m^3 sand with erosion contingency plus 20 000 staged over 50 yrs	>750 (PV 400)	M	M	L	L	M	M
- seawalls	full beach 22 000 t rock/ partial 200 m wall	1800/450	M	VH	VH	VH	VH	-
- back beach drainage	600 m major drainage	>600	VH	L	L	L	H	H
- groynes/breakwaters	15 000 t rock	1500	L	H	H	VH	VH	M

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.5 Cullendulla Beach

Details of the Cullendulla Beach assessment are given in Chapter 8. Cullendulla Beach is part of a National Park, but coastline recession and beach erosion have the potential to threaten a sewer rising main, telephone cable and access track along the beach. Wave overtopping and inundation of the back beach wetland also occurs during moderate storm/tide events. The assessment of management options found (see also Table 6 and Figures 6 and 6a):

- **Do Nothing** - the present value of likely future infrastructure and environmental damage would be over $600 000.

- **Protective Works** - the cost of relocating the assets now would be around $500 000. Works to protect the infrastructure such as beach nourishment, or construction of a seawall or groynes would cost around $1.4 million, $2.0 million and $1.3 million respectively. Beach nourishment could cost around $500 000 if a local source (such as Cullendulla Creek shoal) could be utilised, but this is unlikely because of the potential environmental impacts.

Based on the above, relocating the assets when required in the medium future would appear to be the preferred option. Nourishing the beach with locally sourced sand would be feasible if environmental impacts could be reduced. Doing nothing until the infrastructure was damaged would be economically feasible but not socially or environmentally desirable or justifiable.

Table 6: Cullendulla Beach Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV of assumed loss over 50 yrs	>600	-	VH	L	H	M	H
Protective Works:								
- relocate assets	move rising main, telephone lines	500	M	M	-	-	M	M
- beach nourishment	130 000 m^3 sand over 50 years	1400	M	M	L	-	H	L
- revetment walls	20 000 t rock	2000	M	VH	VH	-	VH	-
- groynes	10 000 t rock, 10 000 m^3 sand	1300	M	H	H	-	H	M

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.6 Long Beach

Details of the Long Beach assessment are given in Chapter 9. The coastal hazards affecting Long Beach are beach erosion, climate change induced recession and coastal inundation. The problems are most severe around the entrance to Reed Swamp Lagoon. The assessment of management options found (see also Table 7 and Figures 7 and 7a):

- **Do Nothing -** the present value of damages to property and infrastructure (mainly roads) would be around $300 000 over a 50 year planning period.

- **Development Controls** - existing minimum floor level freeboards are adequate for back beach areas but not for properties in the immediate foreshore zone subject to wave impacts where hazard proofing works are estimated at around $300 000. Additional building protection controls and building setbacks in line with the revised limits are required.

- **Protective Works/Dune Management -** ongoing beach nourishment plus drainage works and revegetation over the eastern 800 m of beach to prevent wave overtopping would cost around $1 000 000 over 50 years (Bay Road $450 000, Sandy Place $600 000), with a present day cost of $700 000. A seawall would be over $2.5 million and could cause a loss of sandy beach amenity.

Based on the above assessment, continuing with and strengthening the existing development controls, including minimum floor levels and building setbacks, would appear to be the best option. Beach nourishment, although not economically justified, may be socially desirable (along Bay Road) and could involve the local community.

Table 7: Long Beach Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	300	L	-	L	M	M	H
Development Controls:								
- minimum floor levels	protect 16 residences	300	-	-	-	L	L	M
- building setbacks	see revised hazard lines	-	M	-	-	L	L	M
Protective Works:								
- beach nourishment	20000 m^3 sand over 50yrs	450/600						
	Bay Rd &/or Sandy Pl	(PV 300/400)	-	M	L	M	H	L
- revetment walls	25 000 t rock	>2500	-	VH	VH	H	VH	-

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.7 Maloneys Beach

Details of the Maloneys Beach assessment are given in Chapter 10. There are no major coastal hazards affecting Maloneys Beach, although flooding of the back beach lagoon area and maintenance problems with the beach boat launching ramp are related to storm waves (see Figure 8).

Because of the minor nature of the coastal hazards identified for Maloneys Beach, no major management options have been recommended. Suggested options could include a flood study of Maloneys Creek, including the combined effects of catchment runoff and elevated ocean levels ($40 000), and regular maintenance on the boat ramp ($5000 per year).

1.7.8 Hanging Rock

Details of the Hanging Rock area assessment are given in Chapter 11. The main coastal hazards in the Hanging Rock Creek area are coastal inundation and wave impacts in the immediate foreshore area during major storm/tide events and coastal inundation combined with catchment runoff in low lying areas along Hanging Rock Creek during significant storm/rainfall events. The assessment of management options found (see also Table 8 and Figures 9 and 9a):

- **Do Nothing** - the present value of potential coastal hazard damages would exceed $500 000 over a 50 year planning period, but damages would be much higher if major catchment runoff flooding was also included.

- **Environmental Planning** - the extent of existing development and the large number of developments affected means that options such as voluntary purchase would cost over $5 million and so are unlikely to be economically or socially feasible.

- **Development Controls** - existing minimum floor level freeboards are probably adequate in the immediate foreshore zone (0.5 m) although wave impact controls are required. In backshore areas levels are largely determined by catchment runoff. The cost of implementing development controls, such as house raising or site filling, would be over $1.0 million for a minimum program.

- **Protective Works** - raising and upgrading the existing training wall would prevent most wave overtopping and impact damage but could not prevent inundation because of the navigation openings. The cost would vary depending on the construction method but would be less than $1.5 million. Constructing a levee to protect the caravan park alone would cost around $300 000, while a larger levee around most of the developments threatened by coastal hazards would cost around $750 000.

Based on the above, construction of a levee around the caravan park area only would appear to be the most viable option. A levee around all of the Hanging Rock subdivision may be cost effective if catchment flooding was included in the damages assessment.

Table 8: Hanging Rock Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	present day damages over 50 yrs	500	-	-	-	H	M	M
Environmental Planning:								
- voluntary purchase	not feasible	>5000	M	-	-	M	VH	-
Development Controls:								
- minimum floor levels	new development only	-	-	-	-	L	M	M
- building protection	raise 10 residences	>1000	-	-	-	L	M	M
Protective Works:								
- training wall (raising)	4000 t rock topping or 1400 t widening	500/1500	L	L	-	-	H	-
- wave barriers/levees	fill and drain with local/wider protection	300/750	M	L	L	-	M	L

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.9 Corrigans Beach

Details of the Corrigans Beach assessment are given in Chapter 12. The coastal hazards which impact on Corrigans Beach are beach erosion during major storm events with some climate change induced recession, and coastal inundation of low lying back beach (caravan park) areas during major infrequent storm/tide events. There is also potential for some coastal inundation of residential areas when combined with runoff from the Joes Creek catchment. The assessment of management options found (see also Table 9 and Figures 10 and 10a):

- **Do Nothing** - the present value of the potential coastal hazard damages would be around $250 000 over a 50 year planning period (assuming some changes to park management and not including catchment induced flooding).

- **Environmental Planning** - voluntary purchase is unlikely to be feasible in the short term because of the potential need to purchase and close down one of the caravan parks (cost >$500 000) and the resultant social and economic impacts. However, this option may become feasible and desirable as climate change effects increase.

- **Development Controls** - existing minimum floor levels are determined more by catchment runoff than coastal inundation and appear reasonable. The cost of a program of building protection, including caravan site filling and minimal house raising, would exceed $1.2 million. Planned retreat would help address the erosion problem but not the problem of coastal inundation. Planned retreat could be combined with minimum floor level requirements to provide a full option.

- **Protective Works** - construction of a seawall or beach nourishment with local sand to protect development from inundation and erosion would cost $2.5 million and $650 000 respectively. The cost of a partial seawall to protect against erosion only would be over $1.0 million. Initial beach nourishment to prevent overtopping only would be around $200 000 for local sand and would help avoid bypassing of the entrance training wall.

Based on the above, a continuation of the minimum floor level policy with a planned retreat policy, and no major landfill building protection, would appear to be the most feasible options, although voluntary purchase may become feasible and desirable in the future as climate change impacts increase.

Table 9: Corrigans Beach Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	250	M	-	-	M	M	H
Environmental Planning:								
- voluntary purchase	caravan park & residences	>500	H	-	-	H	H	-
Development Controls:								
- minimum floor levels	new development only	-	-	-	-	-	-	M
- building protection	caravan park and 5 residences	1200	-	M	L	-	VH	M
- planned retreat	caravan park	-	M	L	L	-	-	H
Protective Works:								
- seawalls	full beach 40 000 t rock/ partial 12 000 t	2500/1000	L	VH	VH	H	VH	-
- beach nourishment	full beach 65 000 m^3 local sand over 50 years/partial	650/200	L	M	M	L	H	L

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.7.10 Caseys Beach

Details of the Caseys Beach assessment are given in Chapter 13. The main hazards affecting Caseys Beach are wave overtopping of the existing seawall and the structural stability of the wall revetment under wave attack during major storm/tide events. The assessment of management options found (see also Table 10 and Figures 11 and 11a):

- **Do Nothing** - the present value of the potential coastal damages are around $450 000 over a 50 year planning period plus significant amenity and social losses.

- **Development Controls** - the cost of hazard proofing properties from inundation would be around $100 000 using a wave deflection barrier/roadside gutter, but this would not prevent damage to the existing seawall and road. Ongoing seawall and road repair and maintenance/upgrading is estimated to cost around $150 000 per 10 year event (>$600 000 total over a 50 year period), and have a present value of around $300 000.

- **Protective Works** - reforming of a dune/sandy beach along the seawall would cost around $2.0 million and requires more investigation to assess its engineering feasibility. Reconstructing the seawall to a standard sufficient to prevent damage would cost around $3.0 million. Constructing an offshore breakwater/reef would exceed $5.0 million.

Based on the above, ongoing maintenance/upgrading of the seawall and Beach Road is economically the most feasible solution, and when combined with the construction of a wave deflection barrier, probably provides an adequate social solution.

Table 10: Caseys Beach Management Options Summary

Management Option	Description/ Comment	50 Year Cost ($'000)	Land Ownership & Tenure	Aesthetics & Ecology	Recreational Amenity	Social Issues	Economic Issues	Climate Change
Do Nothing	PV damages over 50 yrs	450	-	-	H	VH	M	M
Development Controls:								
- building protections	400 m wave wall plus road repair	>700 (PV 400)	-	L	L	L	M	M
Protective Works:								
- seawalls	30 000 t rock	3000	-	VH	VH	H	VH	-
- beach nourishment	250 000 m^3 sand	2000	-	H	L	L	VH	L
- offshore breakwaters	50 000 t rock or geotextile bags	5000	-	L	-	L	VH	-

Likely Management Option Impacts: L - low, M - medium, H - high, VH - very high.
PV - Present Value based on 7% discount rate over 50 years.

1.8 Options Recommendations

Based on the coastline hazards evaluation and the assessment of social, environmental and economic impacts undertaken for this Management Plan, the management options as summarised in the following Table 11 are preferred for implementation by Eurobodalla Council and Batemans Bay Costal Management Committee.

Table 11: Summary of Do Nothing and Preferred Management Options

Area	Management Option	Description/Comment	50 Yr PV Cost** ($'000)
CBD	Do Nothing Training Wall Repairs Minimum Floor Levels/Building Protection	Present day damages over 50 years Immediate 3000 t rock plus reshaping Modify 37 commercial, raise 6 residences	1 500 200 1 400
Beach Road (Boat Harbour - West)	Do Nothing Minimum Floor Levels/Building Protection Training Walls	Present day damages over 50 years Raise 10 residences 4000 t rock topping or/and widening	500 400 500/700
Wharf Road	Do Nothing Voluntary Purchase Minimum Floor Levels * Training Walls	Present day damages over 50 years Rationalise foreshore land ownership Not including major landfill Western 4000 t	850 ? - 500
Surfside Beach	Do Nothing Minimum Floor Levels/Building Protection Beach Nourishment	Present day damages over 50 years Modify 20 residences & protect 3 Initial contingency plus 20 000 m^3 over 50 yrs	400 300 400
Cullendulla Beach	Do Nothing Relocate Assets	Assumed loss over 50 years Move rising main, telephone line	>600 500
Long Beach	Do Nothing Minimum Floor Levels /Building Protection Building Setbacks * Beach Nourishment*	Present day damages over 50 years Protect 16 residences As per revised hazard lines 10 000 m^3 over 50 yrs, Bay Rd only	300 300 - 300
Maloneys Beach	Minor Actions	Flood Study & Beach Ramp Maintenance	40
Hanging Rock (Boat Harbour - East)	Do Nothing Levees	Present day damages over 50 years Fill & drainage, local or wider protection	500 300/750
Corrigans Beach	Do Nothing Voluntary Purchase Minimum Floor Levels * Planned Retreat *	Present day damages over 50 years Caravan park Not including major landfill Move first row during re-development	250 >500 - ?
Caseys Beach	Do Nothing Building Protection*	Present day damages over 50 years 400 m wave deflection wall & road repair	450 400

* Partial Option which does not provide full hazard protection.
** Estimated Present Value Costs based on 7% discount rate including both private and public costs.

2. HAZARD REVIEW METHODOLOGY

2.1 Coastal Inundation Review

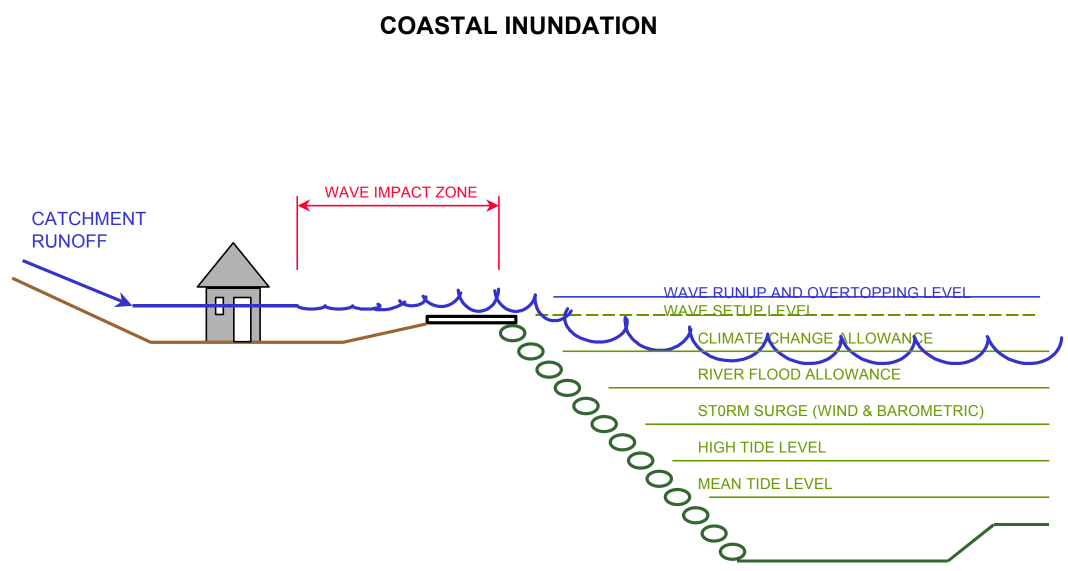
2.1.1 Foreshore Setup Level

The general approach for estimating foreshore setup levels was similar to the method used for estimating "still water" levels in the Vulnerability Study. Each of the significant component parts which make up elevated water levels along the foreshore were examined separately and in combination, based on the likely recurrence of the combined parts. To facilitate later damages assessments, a range of levels for the 1%, 2% and 5% AEP events were determined.

The component parts used for the foreshore setup level assessment were (see also Diagram 1):
- astronomic tide,
- storm surge (wind stress plus barometric setup),
- river flooding,
- climate change allowance,
- wave setup.

Diagram 1: Estimated Inundation Levels

Astronomic Tide and Storm Surge



Astronomic tides are caused by the relative motion of the earth, moon and sun, and their gravitational attractions. Along the NSW coast the mean tidal variation is around ±0.5 m from mean tide level, but variations of ±1.0 m are not uncommon.

Storm surge (wind stress plus barometric setup) is associated with major storm events and onshore winds. Wind stress setup is caused by wind friction creating wind drift currents which setup against the coast. Barometric setup is caused by low atmospheric pressures which can cause water levels to rise by up to 10 mm for each drop in hPa (although this is rarely, if ever, fully achieved).

Because it is an open bay, the astronomic tide and storm surge are general water level components which affect the entire Batemans Bay study area approximately equally.

Fort Denison is an open estuary/ocean gauge location with a very long record which provides an accurate historical record of storm tide levels for open waters on the NSW coast. Recorded water levels at Fort Denison between 1914 and 1990 show that the estimated combined astronomic tide and storm surge for the 1%, 2% and 5% AEP events in an open estuary on the NSW central coast are 1.50, 1.47 and 1.43 mAHD respectively (AWACS, 1991). An analysis of the tidal records for Fort Denison and at the Princess Jetty in Batemans Bay shows that the bay astronomic tide is at least 95% of the open water ocean tide (MHL, 1995).

An analysis of storms which have affected the NSW coast over much of this century (BB&W, 1985) shows that there are slightly fewer and less severe storms, and a less severe wave climate, on the south coast when compared to the central coast (Fort Denison). Analysis of water levels in Batemans Bay during the late 1980's (MHL, 1990) showed that water level anomalies (the difference between predicted astronomic tides and actual recorded tides) were greater at the Princess Jetty recorder than in Port Jackson. The report concluded that the increased anomalies were probably due to Clyde River flooding and local wind setup along the CBD foreshore.

Based on the above, open water astronomic tide plus storm surge conditions in Batemans Bay are similar, but possibly slightly less than for Fort Denison. Design storms would therefore reasonably produce a general astronomic tide and storm surge level in Batemans Bay of around 1.50, 1.45 and 1.40 mAHD for the 1%, 2% and 5% AEP events respectively.

River Flooding Levels

The foreshore water level analysis also included consideration of possible Clyde River flooding effects on bay levels. The approach used for the Vulnerability Study was adopted. This approach utilised rainfall runoff and a hydrodynamic model to predict flood heights for a range of rainfall events in the Clyde River (PWD, 1989).

The investigation found that the probability of major river flooding coinciding with peak storm and maximum storm surge conditions was high, but not mutually dependent. Because of the likely coincidence, a 5% AEP river flood allowance was included with the 1% AEP storm surge level. However, unlike the storm surge level the estimated flood level was varied depending on the location, with a maximum increase of 0.1 m for the 5% AEP flood immediately downstream of the

Princes Highway Bridge, but with increases of less than 0.05 m around the remainder of the inner bay, and zero elsewhere.

Climate Change

An allowance for "Climate Change" was also included in the foreshore level assessment. Based on climate modelling, increased Greenhouse gasses are predicted to cause a rise in ocean levels of between 0.07 m and 0.39 m over a 50 year period. The estimated "most likely" increase according to the United Nations Intergovernmental Panel on Climate Change is 0.2 m by the year 2050 and 0.5 m by 2100 (IPCC, 1996).

The Batemans Bay CMC has adopted a fifty year planning period and for the inclusion of a 0.2 m "most likely" climate change surcharge level in assessments for the Batemans Bay Coastline Hazards Management Plan. Note, the 0.2 m Climate Change would develop progressively towards the end of the 50 year planning period, and does not affect current or past events or conditions.

The increase in Greenhouse gases is also predicted to change weather patterns. The predictions are preliminary and variable, but generally indicate a likelihood for increased storminess. The possibility of increased storminess has been considered in the review of coastal hazards and in the assessment of management options in terms of their flexibility and robustness.

Wave Setup

The final component used in the assessment of foreshore setup levels was an allowance for wave setup on the foreshores. This assessment was based on the wave refraction work undertaken for the Vulnerability Study, plus more recent data studies on wave setup along the NSW coast.

Nearshore wave climate within Batemans Bay is affected by the shoals and sand flats in the bay and by the Clyde River entrance channel along the foreshore. As a result, ocean waves entering the bay are modified by shallow water and friction effects such that major storm waves with offshore significant wave heights of over 7 m (5% AEP) become nearshore waves of around 1.0 to 2.0 m (PWD, 1989).

Wave setup mainly develops in the breaker zone as the wave thrust decreases causing mean water levels to rise. Against a barrier such as a shoreline the increased water levels cannot dissipate, resulting in wave setup. A study of wave setup on natural beaches along the NSW coast (Hanslow & Nielson, 1993) showed that wave setup is largely confined to the nearshore area inside the breaker zone and is highly dependent on the wave height, beach slope and embayment shape.

More recent studies (Hanslow et al, 1996) showed that wave setup is very small at estuary entrances because there is no barrier to prevent dissipation. Wave setup is also substantially relieved by along shore currents when waves approach the shoreline at an angle or by foreshore overtopping (if the potential setup level exceeds the back beach dune crest or training wall level).

It follows from the above studies that wave setup would occur along Batemans Bay foreshore areas, but would be relieved by both the presence of the estuary entrance and the entrance channel, and by the fact that ocean waves often impact on the foreshores at an angle. Further, because foreshore crest levels are often quite low, wave setup would only occur to the crest level. Above that level, waves overtop the foreshore and flood backshore areas.

2.2 Wave Overtopping and Inundation

2.2.1 Wave Runup and Overtopping

To determine wave overtopping rates along bay foreshores during design storm conditions, the procedure identified in the Shore Protection Manual (CERC, 1984) was used. This method is based on theoretical wave overtopping, including a wave setup allowance. The method assumes regular waves and gives calculation procedures for a range of wall slopes and surface roughness conditions.

An assessment was also made of theoretical wave runup levels should the foreshore be raised as part of any future mitigation works. This assessment assumed the foreshore would be increased in height using the same material and slope as the existing foreshore. These heights can be reduced by using different materials and slopes, or if appropriate wave deflection/recurve walls.

2.2.2 Inundation Assessment

To assess the level of flooding from combined local runoff and coastal inundation, a hydrodynamic model was set up of the worst affected area, the CBD area (Appendix A). This model included a hydraulic modelling component (IEA, 1987) which was used to determine runoff hydrographs (flows) for a range of localised rainfall events including the 1%, 2% and 5% AEP design events.

The modelling showed that during a 1% AEP design storm event very large flows entered the CBD from the bay (through the high tide period) compared with those entering from the catchment. Indeed, maximum inundation levels were largely determined by the very large flows entering the area from the bay as a result of wave overtopping.

A similar assessment procedure to that used for the CBD area was then adopted for the other areas with major overtopping flows. This allowed a reasonable inundation level to be calculated without the need for extensive numerical modelling.

2.3 Shoreline Recession and Beach Erosion

2.3.1 Shoreline Recession

To determine shoreline recession a comparison of June 1942 and April 1993 aerial photography was undertaken as part of the Vulnerability Study. This photogrammetry over a 51 year period showed the distance the shoreline had moved, or the volume change above a certain height (usually around 1 mAHD). Shoreline recessions for various sections of beach were then estimated, based on an average of the maximum recessions along that section.

Where possible, the observed changes to the beach profile were associated with known events such as training wall extension, dredging, beach nourishment, and/or the flood/non-flood cycle which affects shoaling patterns in the bay and hence the refracted wave climates at the various beaches.

The contribution to shoreline recession from a medium climate change related sea level rise of 0.2 m over the next 50 years was also calculated in the Vulnerability Study using the Per Bruun Rule (Bruun, 1981). This method effectively erodes the dune shoreline until the beach profile in the near shore zone is raised by the same amount as the sea level change.

The method is probably conservative (an overestimate of recession) for bay beaches where there is considerable bed sediment movement and supply associated with estuary process (tidal and fluvial flows) as well as coastal wave/littoral processes (WBM, 1999). However, given the uncertainties, the recession rate was generally considered to be appropriate.

2.3.2 Beach Erosion

An analysis of aerial photography and beach profiles was undertaken for the Vulnerability Study to determine the beach erosion hazard level. The erosion hazard volume, or “design storm bite” was based on a beach erosion envelope determined from comparison of up to 10 sequential aerial photographs covering the period from June 1942 to April 1993.

The procedure adopted for the Vulnerability Study made no allowance for ongoing recession, nor for possible inherent storm bite variations along a beach. Because of this the erosion hazard volume was re-assessed using the previous photogrammetric data and more recent additional surveys undertaken by Council. Where possible external factors affecting the resultant storm bite assessment were considered such as bay sedimentation/coastal processes or likely human impact.

2.4 Sediment Mobility

2.4.1 Sediment Movement Process

There are at least three conceptual models of sediment movement for Batemans Bay (SKP, 1986, PBP, 1997 and WBM, 1999). The first two models are based on the distribution of the different sediment types around the bay and morphological information from hydrosurveys and aerial photographs. The latest WBM model includes considerations based on indicative uncalibrated 2D modelling of wave induced sand transport.

All the models identify similar main sediment movement processes, although the WBM model suggests a more clockwise movement of sediments in the inner bay Surfside/Cullendulla Beach embayment under catchment flooding and storm wave conditions (see Diagram 2 modified extract from PBP, 1997).

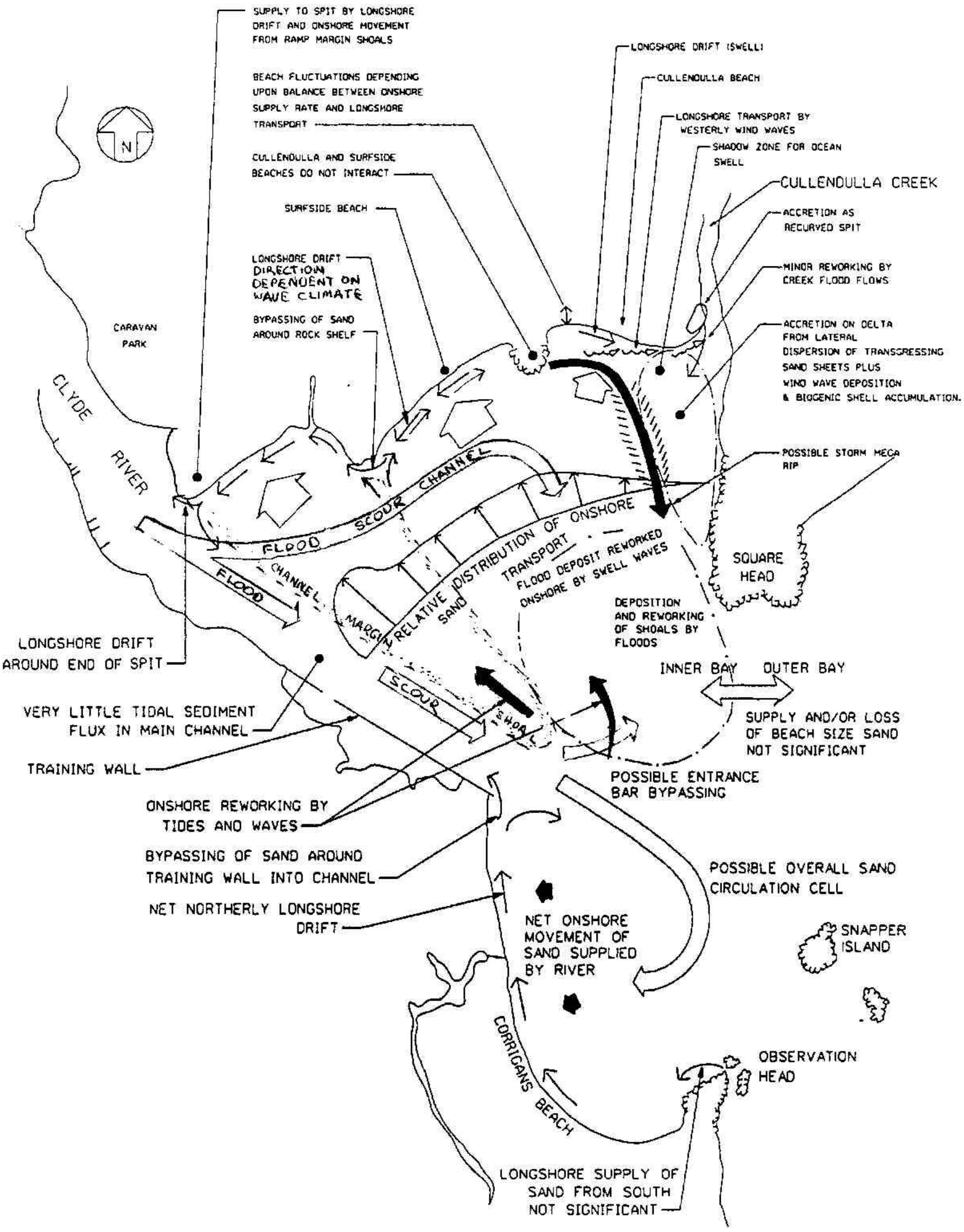
In the inner bay area sediment movement is dominated by a cycle of scour during major Clyde River flooding, and wave and tide induced erosion and deposition during non-flood periods.

During major floods sand is scoured not only from the main channel but also from the Wharf Road/channel margin shoal area. Sand from the main channel is deposited on the entrance bar and off Corrigans Beach. Some finer sediments are also carried further offshore. Sand from the channel margin shoal is deposited into the region between Surfside Beach and Square Head.

After floods, the sands deposited on the bay bed are slowly reworked back towards the shore by waves and tidal currents. This onshore movement gradually depletes sand from the bay bed nearshore zone along Cullendulla and Surfside beaches, and rebuilds the channel margin shoal along the northern edge of the river channel as well as Corrigans, Surfside and Cullendulla beaches.

Available hydrosurvey and aerial photograph evidence (refer PWD, 1989; CRMD, 1996; WBM, 1999) indicates that the present flood/non-flood cycle commenced in the early 1970's (possibly after flooding in 1970, 74 and/or 75), and is now well developed. There also appears to have been a well developed cycle in the latter part of the 19th century possibly ending with the 1909 flood, and another through the 1950's with only partial foreshore development possibly disrupted by several significant floods including the major 1952 and 1955 floods. Another partial cycle then ran through to the 1970's when the present cycle commenced.

Diagram 2: Conceptual Sediment Movement Model



Source: PBP, 1997 (modified)

During the non-flood period, secondary tide channels form through the channel margin shoal, particularly along the Wharf Road foreshore. The existence, location and size of these channels varies in response to wave induced movements and tidal and flood flows. The presence or otherwise of the Wharf Road channel determines the supply of sand and the shape of the Wharf Road and western Surfside Beach foreshores.

The foreshore profile of the remainder of Surfside Beach, Cullendulla Beach and the other bay beaches are determined more by the flood resupply condition of the nearshore beach zones. The movement of sand on to and off the beaches is largely related to wave climate conditions and any associated alongshore currents. The direction and magnitude of these movement is highly complex and varies depending on prevailing storm conditions and shoal morphology during the storm event.

2.5 Revetment Stability

The analyses of revetment stability undertaken for this study were based on the procedure outlined in the Shore Protection Manual (CERC, 1984). This procedure allows for calculation of required/design armour rock size based on the size of the design wave, the slope of the wall and density of the rock, assuming around 5% rock displacement during design storm conditions.

The method can also be used to estimate the wave height which would produce 5% displacement levels on a revetment wall as actually constructed. The ratio of the actual and the design wave heights can then be used to calculate the likely level of damage to the seawall during a design storm.

3. ANALYSIS OF RECENT STORMS

3.1 Design Storm Assessment

Based on an analysis of severe storms along the NSW coastline since 1880 (BB&W, 1985), and an analysis of elevated ocean levels at the Shoalhaven River entrance (L&T, 1987), the design storm at Batemans Bay will probably be associated with a high spring tide, plus a reasonably intense east coast low pressure system off the NSW South Coast.

To create the design storm (1% to 2% AEP) conditions, the low will probably be a Southern Secondary Low, an Inland Trough Low or an Easterly Trough Low, and have a minium barometric pressure at Batemans Bay of around 990 hPa, or around 30 hPa below the long term average pressure. Maximum wind gusts on the coast will be over 120 km/h from the east to south east so as to maximise regional and local wind setup effects, and maximise the offshore wave climate.

A data search of past storm events in Batemans Bay (PWD, 1989) included accounts of waves overtopping the CBD foreshore and “occasionally” washing over Beach Road near the Soldiers Club (where the road is around 1.7 to 1.8 mAHD). This overtopping has occurred for events with a recurrence interval estimated to be less than the assessed 5% AEP event. Note, the analysis method used to assess hazard impacts and damages for this study also includes a climate change allowance of 0.2 m which is not applicable for existing (or past) water levels and storm events.

More recent storm events which have caused notable coastal impacts include the May 1997, the June 1998 and the August 1998 storms. The following analysis compares these storms with the design storm event, based on available wind, wave, tide and barometric pressure data provided by the Bureau of Meteorology and NSW Public Works and Services (MHL) for the NSW coast.

3.2 Actual Storm Analysis

3.2.1 May 1997 Storm

The May 1997 storm was the result of an east coast low which formed over the Tasman Sea on 9 May. The low pressure system then intensified, forming a strong pressure gradient between the low and a high pressure system west of Tasmania on 10-11 May. The centre of the low pressure system focussed over the lower-north coast of NSW between Newcastle and Forster with a minimum barometric pressure of around 1000 hPa.

Recorded data for the Batemans Bay/Moruya area are as follows:

- maximum wind gusts at - Batemans Bay = 37 km/h (SSW & S),
  - Moruya Heads = 65 km/h (SSW),
- minimum barometric pressure at Moruya Heads = 1020 hPa,
- wave Direction (at Sydney) E to SE,
- predicted maximum astronomic tide level at Princess Jetty = 0.76 mAHD on 10/5/97 at 2352,
- recorded maximum water level at Princess Jetty = 1.08 mAHD on 10/5/97 at 2345,
- resultant tidal anomaly = 0.32 m,
- maximum tidal anomaly at Princess Jetty = 0.38 m on 11/5/97 at 2000,
- wave climate:

Location	Hsig (m)	Date	Time	Hmax (m)	Estimated ARI (yr)
Sydney	8.4	10/5	2300	13.8	60
Port Kembla	8.4	10/5	2300	13.8	54
Bateman's Bay (1)	Say 7.3				35
Eden	5.9	10/5	2200	10.7	2

Note: (1) incomplete record.

At Batemans Bay the storm was a significant event in terms of the offshore wave climate with a recurrence interval estimated to be around 35 years (3% AEP). However, the storm was considerably north of Batemans Bay and not as intense as the design storm event, with considerably weaker barometric pressure and lower wind speeds in an offshore/shore parallel direction. These factors would have significantly reduced the effect of the storm on bay water levels.

Based on the analysis undertaken for the Vulnerability Study, the astronomic tide during the storm peak was at or slightly less (around -0.1 m) than would be anticipated for a design storm event, but the barometric pressure and the wind setup effects would have been much smaller (around -0.3 m). As a result the recorded storm surge plus flood level at the Princess Jetty was only 1.08 mAHD, a level which is exceeded several times per year.

Because the general water level was low, wave diffraction and energy losses on the bay shoals would have been much higher than for the design storm. As a result, the foreshore wave climate and wave setup would also have been much lower. Based on the above, the combined recurrence interval of the May 1997 storm wave and water level event would probably be around 10 years (10% AEP) or less.

3.2.2 June 1998 Storm

The June 1998 storm was the result of a low pressure system which moved towards the south coast of NSW from the south-east on 22 June. The low pressure system then intensified, forming a strong pressure gradient between the low and a high pressure system over south western Australia on 23-24 June. The low pressure system focussed over the southern NSW and Victorian coast below Eden with a minimum barometric pressure below 990 hPa. The low pressure system moved south-east away from the coast on 25 June.

Recorded Data for the Batemans Bay/Moruya area are as follows:

- maximum wind gusts at - Batemans Bay = 28km/h (SW),
  - Moruya Heads = 33km/h (SSW & WSW),
- minimum barometric pressure at Moruya Heads = 990 hPa,
- wave direction (at Sydney) mainly southerly,
- predicted maximum astronomic tide level at Princess Jetty = 0.93 mAHD on 23/6/98 at 2000,
- recorded maximum water level at Princess Jetty = 1.25 mAHD on 23/6/98 at 2000,
- resultant tidal anomaly = 0.32 m,
- maximum tidal anomaly at Princess Jetty = 0.51 m on 23/6/98 at 0700,
- wave climate:

Location	Hsig (m)	Date	Time	Hmax (m)	Estimated ARI (yr)
Sydney (1)	>3.11	26/6			
Port Kembla	3.50	26/6	0400		<1
Bateman's Bay (1)	Say 6.0				5
Eden	6.45	23/6	0700	11.8	4

Note: (1) Incomplete record.

The storm was south of Batemans Bay and although the barometric pressure was quite low the wind direction was south westerly and maximum speeds were low. As a result, the barometric pressure effect was probably around the level estimated for the design storm but the wind stress effect would have been much lower (around -0.15 m). The maximum storm surge occurred when the peak astronomic tide was around +0.5 mAHD, much lower (around -0.35 m) than the anticipated design storm level.

During the next high tide the predicted level was +0.93 mAHD (or around 0.08 m greater than anticipated for the design storm). However, by this time the storm surge had abated and the resultant tidal anomaly was only 0.32 m. This was still sufficient to produce very high water levels in the bay (+1.25 mAHD) with a recurrence of around 10% AEP.

In relation to wave climate the storm was not a particularly significant event (with an estimated recurrence of around 5%) although wave heights of around 3 to 4 metres did persist for several days. Because of the low wave climate wave setup and overtopping of the bay foreshore was not significant.

Based on the above information the combined recurrence interval of the storm wave and water level event would have been less than five years (<20% AEP).

3.2.3 August 1998 Storm

The August 1998 storm was the result of an east coast low which formed over the Tasman Sea on 6 August. The low pressure system then intensified, forming a strong pressure gradient between the low and a high pressure system over southern Tasmania and Central Australia on 7-8 August. The low pressure system focussed over the mid-eastern coast of NSW around Wollongong with a minimum barometric pressure of around 995 hPa. The low pressure system moved east away from the coast on 9-10 August.

Recorded data for the Batemans Bay/Moruya area are as follows:

- maximum wind gusts at - Batemans Bay = 37km/h (S),
  - Moruya Heads = 46km/h (ESE),
- minimum barometric pressure at Moruya Heads = 1009 hPa,
- wave direction (at Sydney) S and SE,
- predicted maximum astronomic tide level at Princess Jetty = 0.84 mAHD on 7/8/98 at 2020,
- recorded maximum water level at Princess Jetty = 1.07 mAHD on 7/8/98 at 2020,
- resultant tidal anomaly = 0.24 m,
- maximum tidal anomaly at Princess Jetty = 0.35 m on 8/8/98 at 0300,
- wave climate:

Location	Hsig (m)	Date	Time	Hmax(m)	Estimated ARI (yr)
Sydney (1)	4.97	8/8	0000		0.5
Port Kembla (1)					
Bateman's Bay (1)	Say 6.5				11
Eden	5.59	8/8	0000	8.8	1

Note: (1) Incomplete record.

The storm was north of Batemans Bay with a moderately low barometric pressure and winds from the south with low to moderate gust speeds. Because of these factors storm surge and local wind effects would have been much less than anticipated for the design storm (around -0.4 m), although the astronomic tide was around the level anticipated for a potential major overtopping event.

The storm was a reasonably significant events in terms of wave height, although it was a much smaller event than the May 1997 storm. However as discussed previously, the smaller waves and lower water levels would have significantly reduced wave setup and overtopping along the bay foreshores.

As a result of the above factors the combined recurrence interval, and hence the impacts of the storm, would have been less than for five year event (<20% AEP). However, the storm did follow closely behind the June storm, and as a result the beaches would have been subject to increased cumulative impacts.

4. CENTRAL BUSINESS DISTRICT

4.1 Coastal Hazard Review

The Central Business District (CBD) area (see Figure 1) was identified in the Vulnerability Study (CRMD,1996) as being at substantial risk from *coastal inundation* combined with *local catchment runoff*.

The CBD is located on the southern foreshore and is part of the inner zone of Batemans Bay. A dumped rock training wall runs along the foreshore from the Princes Highway eastwards (see Figure 2). For much of its length the rock wall is topped by a walkway which varies in level between 1.7 and 2.2 mAHD, and most of the CBD area is at or below the level of the training wall. Drainage for the CBD consists of a partially concrete-lined creek with three large box culverts at the river entrance, as well as seven pipes through the training wall.

The occurrence and magnitude of elevated ocean levels, the coincidence of elevated ocean levels with local catchment runoff, climate change and the effects of waves on the CBD were reviewed for this study. An assessment of flood damages to the CBD was also undertaken.

4.1.1 Foreshore Setup Levels

Elevated foreshore setup levels for the CBD were reviewed based on the approach outlined in Chapter 2. As is common for all areas in Batemans Bay, the same astronomic tide and climate change allowance were applicable throughout. However, because of the inner bay location near the estuary entrance, the allowance made for river flooding was 0.1 m, the estimated amount for a 10 year (10% AEP) flood event.

The nearshore wave climate along the CBD is affected by the shoals and sand flats in the bay. As a result major storm waves with offshore significant wave heights of over 10 m become nearshore waves of around 1.0 m (PWD, 1989).

Wave setup along the training wall in the CBD area would be relieved by both the presence of the estuary entrance and the entrance channel, and by the fact that ocean waves impact on the training wall at an angle. Further, because the crest level of the seawall (1.7 to 2.2 mAHD) is at or slightly higher than the 5% AEP ocean storm surge level plus flood and climate change allowance (1.7 mAHD), wave setup would occur to the level of the wall. Above that level waves would overtop the wall.

Based on the wall height and the available storm wave information, a maximum setup amount of 0.2 m has been adopted for storm waves with a recurrence of less than 5%. When combined with the design storm surge and a climate change allowance, the total setup levels adopted for the training wall near the CBD were 2.0, 1.9 and 1.8 mAHD for the 1%, 2% and 5% AEP storm events (see Table 12).

Table 12: Summary of CBD Foreshore Setup Levels (mAHD)

Location	AEP	Astronomic Tide & Storm Surge	Plus River Flooding	Plus 50 yr Climate Change	Plus Wave Setup	Cumulative Level
CBD	5%	1.4	0.1	0.2	0.1	1.8
	2%	1.45	0.1	0.2	0.15	1.9
	1%	1.5	0.1	0.2	0.2	2.0

4.1.2 Wave Overtopping and Inundation

The adopted setup levels are similar to or higher than the existing CBD foreshore wall, and the surrounding roadways and ground level. Therefore during design conditions, waves would overtop the training wall and encroach over the foreshore and backshore areas.

To determine wave overtopping rates, the modelling procedure identified in Chapter 2 was used. The modelling showed that during a 1% AEP design storm event, flows of up to 360 m^3/s would enter the CBD from the bay and 40 m^3/s would enter from the catchment. Under these conditions the water level in the CBD would be around 2.2 mAHD, with a level in the immediate foreshore wave zone of 2.4 mAHD (see Figure 2).

These levels were largely determined by the very large flows entering the area from the bay as a result of wave overtopping, and would probably last for about an hour (through the high tide period). The results are shown in Table 13. The table also gives the theoretical wave runup should the training wall be raised as part of any future mitigation works.

Table 13: Summary of Wave Overtopping and Theoretical Runup Levels

Location / Wall Length (m)	AEP	Overtopping Flow (m^3/s)	Theoretical Runup (m)	Estimated Wave Overtopping Inundation Levels	
				Backshore (mAHD)	Foreshore (mAHD)
CBD	5%	240	1.1	2.0	2.2
	2%	300	1.1	2.1	2.3
300m	1%	360	1.1	2.2	2.4

Catchment runoff with high tide levels, but without wave overtopping also has the capacity to cause some local flooding in the vicinity of the Soldiers Club, Flora Crescent and Beach Road. This flooding results from the capacity of the box culvert under the Soldiers Club being exceeded. However, this flooding is not as severe as the design ocean inundation levels.

4.2 Coastal Hazard Impacts

4.2.1 Coastal Inundation and Catchment Runoff

Road centreline levels within the CBD rarely exceed 3.0 mAHD, with the lowest surveyed gutter invert level being 1.22 mAHD at the intersection of Orient Street and North Street. Therefore, during a 2% AEP storm, this intersection would be subject to a water depth of almost 0.9 m.

Approximately 30% of buildings in the CBD have a floor level less than the 2% AEP inundation level. The lowest surveyed floor level in the CBD area is 1.63 mAHD (MSB/Fisheries building). During a 2% AEP storm, this building would be inundated by water nearly 0.5 m above floor level. Buildings along the foreshore walkway are also exposed to wave impact hazards and increased water levels, due to wave runup at the buildings. Future development in this area should make allowance for wave impact hazards.

4.2.2 Inundation Damages Assessment

To assess the level of inundation damages a survey of properties in the affected area was undertaken together with a damages analysis. This assessment is based on similar flood inundation studies undertaken in NSW, and estimates damages to residential and commercial buildings based on floor levels and depth of inundation (Appendix B). To further assist with estimating damages to residential and commercial buildings a survey of inundation liable residences and businesses was undertaken. A lump sum infrastructure damages allowance was also added based on the estimated cost of repair or replacement of damaged public infrastructure and utilities (roads, telephone cables, street plants, etc.).

The inundation damages for Batemans Bay CBD and the total damage figures are presented in Table 14.

Table 14: Inundation Damages for Batemans Bay CBD

Event (AEP)	Inundation Level (mAHD)		Damages ($)		
	CBD	Foreshore	Residential / Commercial	Public Infrastructure	Total
1%	2.2	2.4	1 500 000	1 200 000	2 700 000
2%	2.1	2.3	900 000	900 000	1 800 000
5%	2.0	2.2	600 000	600 000	1 200 000

The total damages figure in a given event quantifies the magnitude of the inundation problem. However, when considering the economic effectiveness of a proposed mitigation option the key factor is the total damages prevented over the life of the option. This is a function not only of the high damages which occur in large events but also of lesser (but more frequent) damages which occur in small events.

The standard way of expressing inundation damages is in terms of *Average Annual Damages* (AAD). These are calculated by multiplying the damage that can occur in a given event by the probability of the event occurring in a given year. These numbers are then summed across the range of events. By this means the smaller, more frequent events are given a greater weighting than the rare, catastrophic events.

The AAD for the study area are estimated to be approximately $110 000 up to the 1% event, excluding intangible damages such as the effects of trauma or loss of life. This value was determined from the estimated damages total for the 1%, 2% and 5% AEP's and assuming that no damages occur at and below the 20% AEP.

4.3 Options Assessment

Based on the above review of coastal hazards and the assessment of potential flood damages in the CBD, a number of options to mitigate the potential adverse impacts were assessed. These options included:

- Do Nothing,
- Environmental Planning:
  - restrictive zoning,
  - voluntary purchase,
- Development Controls:
  - minimum floor levels,
  - building protection (raising floors and hazard proofing),
- Protective Works:
  - training wall (repairs),
  - wave barrier/levee,
  - training wall (raising).

4.3.1 Do Nothing and Training Wall Repairs

As stated above, approximately 30% of buildings in the CBD have floor levels below the 2% AEP design water level, and the estimated Annual Average Damages (AAD) are $110 000. Thus, whilst this option involves no cost to implement, on average $110 000 will be incurred in damages each year. This cost excludes intangible factors such as trauma felt by CBD occupants and residents. The present value of the damages (excluding intangibles) assuming a 7% discount rate over a 50 year planning period is $1.5 million.

The extent and severity of the risk, together with the social importance of the CBD area within the region, suggests that “do nothing” is not a viable option. Further, the existing wall is in need of repairs and additional armour in places as a result of past storm damage. The cost to restore the wall to a safe functional condition including reshaping areas of slumping, plus an additional 2000 tonnes of rock, would be around $200 000. Because this cost is common to all the options it has been included as a separate Protective Works item in the assessment.

4.3.2 Restrictive Zoning and Voluntary Purchase

While environmental planning options such as restrictive zonings or voluntary purchase are theoretically suitable for the CBD area, restricting the level or type of development would probably lead to claims for compensation from existing landholders. Further, the voluntary purchase of valuable waterfront commercial properties would requiring tens of millions of dollars.

In practice the cost of implementing these options would be very many times greater than the $1.5 million present value damages identified above. Because of the likely economic cost and obvious social disruption within the CBD, these options have not been considered further.

4.3.3 Minimum Floor Levels

Council already has an interim coastal flooding standard for the CBD which is based on the 1% AEP ocean inundation level plus a 5% AEP river flood allowance (ESC, 1997). This standard sets minimum commercial floor levels in the foreshore areas of 2.6 mAHD, with a requirement to consider wave impacts in those areas immediately facing the bay. In areas away from the bay, where ground levels are higher and there is more residential development, a minimum floor level of 2.85 mAHD has been set.

The implementation of this policy to new development within the CBD reduces the level of damages during a major flood event. A feature which makes this option attractive is the minimal capital cost.

Disadvantages of this option are:
- does not address the hazard to existing development,
- increases development costs within the affected area,
- can create access difficulties for commercial premises which prefer street level entrances,
- can result in large discrepancies between the floor levels of new development and adjacent existing buildings.

Based on the water levels determined for this study (see Table 13 and Figure 2) the existing interim flooding policy would provide a freeboard allowance of 0.2 m in the immediate foreshore zone subject to wave impacts. This allowance would be exceeded by wave runup in places and so would need to be combined with hazard proofing of individual properties. In the remaining foreshore areas the freeboard allowance is 0.4 m.

4.3.4 Building Protection

Another option, which complements the minimum floor level policy would be for Council to establish development control standards which require proposed development to be designed to withstand the assessed coastal hazards. This option has the advantage of focussing on individual at-risk developments and providing protection where it is needed. The primary measures would involve raising floor levels, sealing openings, redirecting flows and/or constructing physical barriers on a development-by-development basis.

The policy would target new development and major redevelopment, but also help address problems for existing development through a program of house raising and hazard proofing. The affected properties are mainly those backing on to the foreshore area along Clyde Street, Orient Street and Beach Road, but also includes properties facing the foreshore along Clyde Street and Beach Road and low lying properties backing on to the wetland behind Flora Crescent and Orient Street (see Figure 2a).

There are limitations to the types of structures that can be raised (generally only non-brick, single storey houses on piers). Further, this option does not address the continuing potential damages to infrastructure and property not raised (such as garages, pools, laundries and gardens).

The protection requirements for each individual property will need to be more clearly identified, but possible advantages include:

- not protecting a larger area than necessary,
- apportioning costs more readily, and
- avoiding construction of a structure along the foreshore.

The cost to hazard proof an individual commercial property is typically around $30 000, with approximately $40 000 required to raise a timber house above the level of inundation. The total cost of this option, based on the 37 flood affected commercial properties and 6 residences, would be around $1 400 000. If this option were restricted to those properties in the immediate foreshore zone subject to wave impact/runup, the cost of this option would be around $600 000.

4.3.5 Wave Barrier/Levee

Construction of a wave barrier along the existing foreshore training wall could be undertaken without significant impact on foreshore use or development. A typical wave barrier could consist of a continuous concrete wall on the bay side of the walkway, with a crest level of around 3.0 mAHD, which would make it around 1 m high (see Figure 2a). The wall would be curved on the bay side to reflect waves. On the inside the wall could be stepped or vertical as appropriate. Improvements would be required to the wall between the Princes Highway and Innes Boatshed. To tie the barrier into high ground, Beach Road would have to be raised east of the Soldiers Club culverts.

This option would also require the existing drainage to be sealed against ocean inflows. The construction of flood gates on the drainage outlets would be a major part of this work (see Figure 2a). However, because of the possibility of internal flooding due to high local catchment runoff, construction of a large pumping station or alternative drainage methods, such as diverting flows westwards to the estuary, would also need to be considered.

Based on the available information, the estimated cost of constructing a seawall/wave deflection barrier along the foreshore from the Princes Highway Bridge along the foreshore to link in with and including a raised section of Beach Road west of the Soldiers Club culverts would be around \$850 000 and the construction of flood gates on drainage outlets would cost approximately \$350 000. Alternative drainage works to direct flows or a pumping station would probably exceed \$500 000, but would provide a more reliable mitigation strategy.

4.3.6 Training Walls

As an alternative to constructing a wave barrier along the existing foreshore wall it would be possible to raise the wall by extending the rock revetment to a height of around 3.0 mAHD (see Figure 2a). Armour stone for the wall face of around 1.0 tonnes, and widening and grading the back of the wall base would require around 4000 tonnes of rock and 5000 m^3 of fill, at a cost of around \$600 000.

In the foreshore section between Innes Boatshed and the Soldiers Club, the raised wall would intrude upon the existing foreshore walkway. This would require either a new pathway to be constructed or the foreshore wall extended outwards. Raising and replacing the walkway would impact on neighbouring properties , complicate drainage and is likely to be unacceptable to CBD owners and users. Extending the wall would require at least a further 10 000 tonnes of rock and cost in excess of \$1 000 000.

In addition to these works, it would be necessary to raise a section of Beach Road and install flood gates on the drainage culverts as per the wave barrier option. These additional works would add a further \$500 000 to the option cost resulting in a total cost of over \$2 000 000.

4.4 Options Summary

As indicated above, the main coastal hazards for the area are inundation as a result of waves overtopping of the training wall and wave impacts along the immediate foreshore. The assessment of management options found (see also Summary Table 2 and Figures 2 and 2a):

- **Do Nothing -** the present value of the likely damages would be around \$1.5 million over a 50 year planning period but the social dislocation likely to be caused would make the real costs much higher.

- **Environmental Planning** - options such as voluntary purchase or planned retreat are not feasible because of the extent and value of the existing development and the disruption they would cause to ownership and management of the CBD.

- **Development Controls** - existing minimum floor level freeboards are probably adequate (0.65 m) for residential developments and commercial properties (0.4 m) but are low in the immediate foreshore area (0.2 m) although development in this area is also required to consider wave impacts. The cost of building protection is around $1.4 million. Both options have moderate to low impact on the amenity of the area and its use.

- **Protective Works** - reshaping and additional rock are required to repair the existing training wall at a cost of around $200 000. Construction of a concrete wave barrier/levee along the foreshore with associated road and drainage works would cost around $1.7 million. Alternatively, raising and widening the rock wall would cost in excess of $2.0 million. Both options would impact on the visual amenity of the foreshore area.

Based on the above, continuation of the existing development controls, possibly with some modifications would appear to be the best coastline hazard management option. More detailed consideration of the foreshore wave barrier option could also be considered.

5. BEACH ROAD

5.1 Coastal Hazard Review

The Beach Road area adjacent to the CBD on the southern foreshore of the inner zone of Batemans Bay, encompasses the area west of the Batemans Bay boat harbour/marina (including the marina) and the area west of Hanging Rock Creek (see Figures 1 and 3). The main coastal hazards identified for the area are *coastal inundation* combined with *local catchment runoff*.

A rock training wall runs from the CBD to the boat harbour (see Figure 3). The top of the wall ranges in level between 1.8 and 2.2 mAHD. Nearly all the foreshore is road reserve or open space, at or below the level of the training wall. There is one large residential development on raised ground above flood level, and some minor marina buildings near the foreshore. Beyond Beach Road much of Herarde and Heradale Streets, the eastern section of Golf Links Road, and large parts of the Golf Course are below 2.0 mAHD. These areas are also (primarily) affected by flooding from Hanging Rock Creek.

As for the CBD, the design still water and wave runup levels adopted for the area in the Vulnerability Study are based on theoretical conditions and include an uncertainty factor. Because of this, the estimates for elevated foreshore setup levels and wave climates were reviewed for climate change effects and a range of design events. This information was then used to assess inundation impacts.

5.1.1 Foreshore Setup Levels

Elevated foreshore setup levels for the west of the Boat Harbour were reviewed based on the approach outlined in Chapter 2. The nearshore wave climate along the Beach Road area is affected by the shoals and sand flats in the bay. As a result of the shoals, major storm waves with offshore significant wave heights of over 10 m become nearshore waves of around 1.5 m (PWD, 1989).

Because of the inner bay location near the estuary entrance, the same astronomic tide and climate change allowance are applicable for the area as was assessed for the CBD (see Table 15). Slightly lower river flooding and slightly higher wave setup effects have been adopted because of the more exposed location.

Table 15: Summary of Beach Road Foreshore Setup Levels (mAHD)

Location	AEP	Astronomic Tide & Storm Surge	Plus River Flooding	Plus 50 yr Climate Change	Plus Wave Setup	Cumulative Level
Beach Road	5%	1.4	0.05	0.2	0.15	1.8
	2%	1.45	0.05	0.2	0.2	1.9
	1%	1.5	0.05	0.2	0.25	2.0

Based on the revised assessment, the setup water levels adopted for the area were 2.0, 1.9 and 1.8 mAHD for the 1%, 2% and 5% AEP events respectively.

5.1.2 Wave Overtopping and Inundation

The adopted setup levels are similar to the existing Beach Road foreshore wall, and higher than Herarde Street and the surrounding roadways and the general ground level. Therefore, during design conditions waves would overtop the training wall and encroach over the foreshore and backshore areas.

To determine wave overtopping rates, the procedure identified in the Shore Protection Manual (CERC, 1984) was used (see Table 16). The table also gives the theoretical wave runup should the training wall be raised as part of any future mitigation works.

Table 16: Summary of Wave Overtopping and Theoretical Runup Levels

Location / Wall Length (m)	AEP	Overtopping Flow (m^3/s)	Theoretical Runup (m)	Estimated Wave Overtopping Inundation Levels	
				Backshore (mAHD)	Foreshore (mAHD)
Beach Road	5%	300	1.6	2.0	2.2
	2%	400	1.6	2.1	2.3
300m	1%	450	1.6	2.2	2.4

Based on the assessed levels, wave overtopping of the training wall would occur during high tide, with intense storm conditions, and would probably coincide with high catchment rainfall and runoff. Modelling undertaken for the CBD showed that maximum inundation levels were largely determined by the very large flows entering the western foreshore area from the bay, and that these would last for approximately an hour (through the high tide period). There would also be wave overtopping into the Boat Harbour.

On this basis, the Beach Road/Boat Harbour area was identified as having similar peak wave inundation levels to the CBD (i.e. 2.4 mAHD near the training wall and 2.2 mAHD away from the wall during a 1% AEP event). Note, away from the foreshore, in the Golf Course area, water levels would be additionally affected by flooding from Hanging Rock Creek. The impacts of this flooding would need to be assessed as part of a separate catchment flood study.

5.2 Coastal Hazard Impacts

5.2.1 Coastal Inundation

Beach Road near the CBD and the boat harbour/marina are exposed to wave attack and overtopping flows during major storms with very high tides. Beach Road would need to be closed to prevent damage to vehicles and possible loss of life (some 10 residential and tourist accommodation properties could also be affected). Boats moored/trapped in the boat harbour and boat harbour infrastructure would be particularly susceptible to damage by overtopping waves which exceed the design wave conditions for the moorings. A study of wave climate in the boat harbour (AWACS, 1995) determined the 1 in 100 yr significant wave height to be 0.65 m by combining wind waves and diffracted ocean storm waves for a water level of 1.5 mAHD.

Beyond Beach Road numerous low lying properties could be flooded along Herarde and Heradale Streets and Golf Links Road. This flooding would be a combination of ocean inundation and catchment runoff. There could be some environmental damage as a result of salt water inundation, but this would probably be mitigated by following rain.

5.3 Options Assessment

There are several options and combinations of options available for the Beach Road (western Boat Harbour) area including:

- Do Nothing,
- Environmental Planning:
    - voluntary purchase,
- Development Controls:
    - minimum floor levels,
    - building protection (raising floors and hazard proofing),
- Protective Works:
    - training wall (raising).

5.3.1 Do Nothing

The main advantage of a “do nothing” approach is that it involves no cost to implement. However, there are potentially very large direct and indirect costs associated with future wave overtopping and inundation damage of the area.

Using a similar approach to that used for residential development in the CBD area, tangible damages of around one million dollars could be expected to property and infrastructure during a 1% AEP event. This corresponds to an AAD of around $35 000, with a net present value of $500 000 based on a discount rate of 7% pa and a 50 year planning period.

Note, this figure includes a significant local catchment runoff component, but could be much higher if extensive damage also occurred to boat harbour infrastructure and vessels moored in the marina. Additional investigation should be undertaken at a local level into wave overtopping of the seawall and its potential impact on the marina facilities and the vessels moored in the boat harbour.

5.3.2 Voluntary Purchase

While voluntary purchase is theoretically suitable as a mitigation measure for this area, the number of possible residential and tourist accommodation properties affected by wave overtopping/flooding is large while the likely extent of inundation is low. Because of this voluntary purchase would not be an economically or socially feasible option.

5.3.3 Minimum Floor Levels

Council has an interim coastal flooding standard for the area which is based on the 1% AEP ocean inundation level, plus a 5% AEP river flood allowance, and 0.5 m freeboard (ESC, 1997). This standard sets a minimum residential floor level of 3.0 mAHD over the Beach Road area based on the Ocean Inundation Study (PWD, 1989). This policy has been applied to all new development since 1988.

Based on the water levels determined for this study which do not include runoff from the local catchment, the existing interim flooding policy would provide a freeboard allowance during a 1% AEP event of 0.6 m in the Beach Road foreshore area and 0.8 m in the backshore area. Including local catchment runoff would result in a lesser allowance in the areas additionally affected by catchment runoff.

The implementation of this policy has significantly reduced the potential level of damages during a major overtopping event. However, away from the immediate foreshore area there remain a substantial number of properties built before the policy started with floor levels below the set standard. Further, this option does not address the potential problem of damage to vessels and infrastructure in the boat harbour.

5.3.4 Building Protection

An alternative to purchasing properties and as a complementary measure to the minimum floor level policy, Council could establish development control standards which require proposed development to be designed to withstand the assessed coastal hazards. This option has the advantage of focussing on individual at-risk development and providing protection where it is needed. The primary measure would involve having floors above inundation levels, but may also involve special foundation requirements, wall strengthening, wave runup barriers, flood proofing, etc.

The policy would target new development and major redevelopment, but also help address problems for existing development through a program of house raising and hazard proofing. However, there are limitations to the types of residences that can be raised (usually only non-brick, single storey houses on piers). The estimated cost to raise an average sized house is $40 000.

Based on raising 10 houses, the cost of implementing this option would be $400 000. Note however, this option does not address the combined problem of catchment runoff inundation nor the potential damages to infrastructure and utilities, particularly the risk to traffic and the need to close Beach Road during an overtopping event, nor the likelihood of extensive damage in the marina/boat harbour.

5.3.5 Training Walls

Overtopping and inundation damage to foreshore development and low lying back beach areas may be prevented by constructing a physical barrier. Partial raising of the wall along the western foreshore near the CBD would reduce inundation damages along Beach Road and in the Herarde Street areas, although flooding problems due to catchment runoff would continue.

The existing rock wall could be raised to a sufficient height (3.0 mAHD, at least) with armour stone of around 1.0 to 1.5 tonnes (see Figure 3a). The cost of this part of the option would be around $350 000 requiring some 3000 tonnes of rock.

Subject to further investigation, raising the wall along the boat harbour would help prevent damage to the vessels and the marina infrastructure and may have the highest priority. Assuming the existing wall can be raised in height without widening the wall base, the estimated cost to raise the existing wall along the boat harbour for some 200 metres is approximately $150 000, requiring some 1000 tonnes of rock. However, if the wall were to be widened (in the vicinity of the boatharbour) an additional 3000 tonnes would be required and the cost would increase to around $350 000 .

In total, training wall options could cost $500 000 or up to $700 000 if wall widening was required.

5.4 Options Summary

The main coastal hazard in the Beach Road area is coastal inundation as a result of waves overtopping the training wall. There could also be significant wave impact damage to boats and infrastructure around the boat harbour and combined catchment flooding/coastal inundation problems along Hanging Rock Creek. The assessment of management options found (see also Summary Table 3 and Figures 3 and 3a):

- **Do Nothing** - the present value of the likely inundation damages would be around $500 000 over a 50 year planning period, although this would be much higher if there was extensive damage in the boat harbour area (this requires further investigation at a local level).

- **Environmental Planning** - there is already a large buffer zone and options such as voluntary purchase are not feasible because of the extent and value of the existing development.

- **Development Controls** - existing minimum floor level freeboards are probably adequate for coastal inundation (0.6 m foreshore, 0.8 m backshore). The cost of implementing a program of hazard proofing such as house raising would exceed $400 000, not including the boat harbour area.

- **Protective Works** - the visual and recreational impact would be limited because of the existing wall and park location. Raising the existing training wall to prevent substantial wave overtopping at the boat harbour would cost around $150 000 or $350 000 if widening of the wall was required. To raise the entire wall could cost up to $700 000.

Based on the above, continuing the existing minimum floor level policy would be recommended as would further investigations to determine the likely impact of wave overtopping on the marina/boat harbour facilities and vessels. Raising the entire training wall would also appear to be a feasible option.

6. WHARF ROAD

6.1 Coastal Hazard Review

The Wharf Road area is the northern foreshore inner zone of Batemans Bay opposite the CBD (see Figure 1). The main coastal hazards identified for the Wharf Road area are *coastal inundation* and *sediment movement* (both accretion and erosion).

The area is divided into two parts, a western rock wall section which is immediately east of the Princes Highway Bridge near the entrance and an eastern beach section (see Figure 4 and Diagram 3).

The rock wall has a crest level between 1.5 and 1.9 mAHD and a depth in the main river channel of over 5 m. Adjacent to the rock wall there is a public reserve and a caravan park on leased public land. Beyond the wall there are caravan parks, residential dwellings and some light industry with typical ground levels (including the roadways) of 1.5 mAHD or less.

Along the beach (or eastern) section there are extensive sand flats and shoals. Most of the beach is below 1.0 mAHD. The sand flats and shoals continue out from the beach (for over 2 km) forming a channel margin shoal along the northern edge of the main river channel.

The position of the mean high tide foreshore along this section is highly variable, depending on the pattern of erosion and accretion. As the result of an 1800's subdivision much of the beach and the submerged nearshore area is now privately owned. Over the years there have been a number of proposals to reclaim and develop this land.

The beach is backed by an open space area (partially vegetated), one residential dwelling and McLeod Street (an extension of Wharf Road). Ground levels in this area are typically less than 1.5 mAHD for around 100 m from the foreshore. Levels then rise quickly to over 10 mAHD, except near the eastern corner of the beach where McLeod Creek leads to the designated (SEPP 14) West Surfside wetland.

As for the CBD, design still water and wave runup levels were reviewed for climate change effects and a range of design events. This information was then used to assess inundation impacts. The movement of sediments in the area was also reviewed based on existing information, including the work undertaken for the Vulnerability Study, an earlier coastal engineering study undertaken to assist land zoning (SKP, 1986), and the Batemans Bay/Clyde River Processes Study (WBM, 1999).

6.1.1 Elevated Foreshore Levels

Elevated foreshore levels for the Wharf Road area were reviewed based on the approach outlined in Chapter 2. Because of the similar inner bay location near the estuary entrance, the same astronomic tide, storm surge, river flooding and climate change allowances are applicable for Wharf Road as were assessed for the CBD. However, because of the different wave climate and foreshore condition, the allowance for wave setup and wave runup was assessed separately.

Diagram 3: Inner Foreshore Zone Aerial Photograph

March 1984

Features of Diagram 3:
- channel margin shoal,
- secondary flood/tide channel,
- entrance bar,
- entrance training wall,
- Cullendulla Beach seagrass beds.

Based on the revised assessment, the foreshore level adopted for the Wharf Road area excluding wave effects was 1.6, 1.55 and 1.5 mAHD for the existing 1%, 2% and 5% AEP events, increasing (with climate change impacts over a 50 year planning period) to 1.8, 1.75 and 1.7 mAHD for the 1%, 2% and 5% AEP events respectively (see Table 17).

Table 17: Summary of Wharf Road Foreshore Levels (mAHD)

Location	AEP	Astronomic Tide & Storm Surge	Plus River Flooding	Plus 50 yr Climate Change	Cumulative Level
Wharf Road	5%	1.4	0.1	0.2	1.7
	2%	1.45	0.1	0.2	1.75
	1%	1.5	0.1	0.2	1.8

Note that the adopted design water levels without wave effects (such as wave setup and runup) are similar to, or marginally higher than, the level of the existing Wharf Road foreshore wall, roadways and general ground level along the foreshore. If the foreshore levels were higher, additional wave setup of 0.15, 0.2 and 0.25 m would apply for the 5%, 2% and 1% AEP storm events (Chapter 2). Further, assuming the foreshore was extended either as a rock revetment wall (western section) or as a sand beach (eastern section), the theoretical wave runup would be at least 1.4 m and 1.8 m respectively (see Table 18).

Table 18: Summary of Theoretical Wave Setup and Runup, and Estimated Inundation Levels

Location / Wall Length (m)	AEP	Theoretical Setup Level (mAHD)	Theoretical Runup (m)	Estimated Wave Overtopping Inundation Levels	
				Backshore (mAHD)	Foreshore (mAHD)
Wharf Rd (West)	5%	1.85	1.4	1.8	2.2
	2%	1.95	1.4	1.9	2.3
200m	1%	2.05	1.4	2.0	2.4
Wharf Rd (East)	5%	1.85	1.8	1.8	2.3
	2%	1.95	1.8	1.9	2.4
200m	1%	2.05	1.8	2.0	2.5

6.1.2 Wave Overtopping and Inundation

The nearshore wave climate at Wharf Road is substantially affected by the shoals and sand flats in the bay. As a result of the shoal pattern which existed during the 1998 hydrographic survey, ocean storm waves with offshore significant wave heights of over 10 m were found to become design nearshore waves of around 1.3 m at the beach area of Wharf Road with 1.6 m theoretical wave runup (PWD, 1989). A reassessment of wave climate for the Vulnerability Study using the 1995 hydrosurvey found that changes to the shoal patterns potentially increased nearshore wave heights, but that these changes were less than 0.3 m.

As previously noted, the ground level of the foreshore and back beach area is similar to the estimated foreshore setup level (without wave effects). Therefore, during design conditions waves would overtop the foreshore and wave set up would be minimal. Waves overtopping the foreshore would dissipate over 30 to 50 metres depending on the obstructions and drain either along the creek leading to the wetlands, or along Wharf Road and re-enter the estuary near the bridge.

The rate of overtopping for the beach and the rock wall sections was estimated using the method set out in the Shore Protection Manual (CERC, 1984). Based on this procedure, the estimated 1%, 2% and 5% AEP overtopping rates would be around 0.8, 0.7 and 0.6 m^3/s per metre length of foreshore, or around 320, 280, and 240 m^3/s, with about half entering the existing rock wall section and half entering the beach section.

Inundation levels as a result of wave overtopping would fall over the initial 30 to 50 m from the foreshore as waves dissipate and the waters drain to the river or backshore areas. Including a climate change allowance, but assuming wave setup is minimal (because of foreshore overtopping), the inundation level near the foreshore would be around 2.5 mAHD during a 1% AEP event. This area would also be subject to wave impact and runup problems. Away from the immediate wave zone the inundation level would drop to around 2.0 mAHD (see Table 18).

6.1.3 Sediment Movement Process

As discussed in Section 2.4, sediment movement processes for the Wharf Road/channel margin shoal area are dominated by major flood/non-flood cycles.

During major floods a secondary flood channel is scoured along the Wharf Road foreshore and the sands deposited offshore into the region between Surfside Beach and Square Head. After the floods the sands are reworked back towards the shore by waves, rebuilding the channel margin shoal and eventually Wharf Road beach prior to erosion during the next major flood (see Diagrams 2 and 3).

The Estuary Processes Study, WBM (1999) described two fundamental foreshore configurations for the Wharf Road area:

- a wave dominated shoreline where sand from the channel margin shoal moves into the area and forms a sand spit attached to the foreshore in a direction more or less parallel to the predominant ocean wave crests (see Diagram 3),
- a nearshore current dominated foreshore shape where the foreshore (usually boarded by a secondary tide channel), runs more or less east-west from Surfside Beach through to the river (see Diagram 4).

The accretion which occurred in the Wharf Road area through the 1980's and 90's is part of a wave dominated period when sand moved onto the Wharf Road foreshore. However, indications from historical records are that the current dominated eroded foreshore shape is a more common/likely condition, separated by shorter periods with a wave dominated foreshore shape.

During wave dominated foreshore periods large quantities of sand move onto the Wharf Road foreshore. During current dominated foreshore periods both accretion and erosion can still occur along Wharf Road but at a significantly slower rate. Sand from the channel margin shoal is moved into the Wharf Road channel by waves, and then either towards Surfside Beach or the main river channel by ebb and flood tides. The sand moved towards Surfside Beach can be cycled onto the beach or the Wharf Road foreshore by waves. Under some conditions sand on Surfside Beach can be worked around the rocky point onto the Wharf Road foreshore (see Diagram 4).

6.2 Coastal Hazard Impacts

6.2.1 Coastal Inundation/Wave Impact

The predicted design ocean levels and wave overtopping would occur infrequently, during a high spring tide, with intense storm conditions, coinciding with high catchment rainfall and runoff. The period of wave overtopping would be limited to the high tide period. However, as for the CBD, the volume of water entering the area due to wave overtopping would be sufficient to fill low lying back beach areas.

As a result of storm conditions and near shore waves with significant heights over 1.3 m, the foreshore strip along the Wharf Road area could expect to have waves of at least 0.7 m crossing the wall and impacting on the caravan parks and residences in the area. Wave inundation levels up to 2.5 mAHD would be over 1.0 m above average ground levels and above the floor levels of many buildings. Note, this does not include the serious wave impact damage which would occur to caravans and low lying residences along the foreshore facing McLeod Street and the eastern part of Wharf Road. In the back beach areas away from wave effects the estimated 1% AEP inundation level would be around 2.0 mAHD which is around 0.5 m above typical ground levels (see Figure 4).

6.2.2 Sediment Movement

Based on the available information, sediment movement in the Wharf Road area should continue to be dominated by flood and non-flood cycles. On this basis, the substantial accretion which has occurred in the area throughout the 1980's and 1990's could continue until the beach is higher and more extensive. Alternatively, major flooding in the Clyde River could again scour the deposited sand and recommence the cycle.

To date, the accretion and erosion cycle has not been a significant coastal hazard in terms of property damage because the area is largely undeveloped sand flats the loss of which simply changes the local beach alignment and provides a sediment supply for other areas of the bay. The low key trapping of sand which has been undertaken may have accelerated natural accretion, but would not prevent major flood scour. However, more permanent trapping of sand or obstructions to flows such as construction of a seawall could change the flood/non-flood cycle and may impact on the sediment supply to Surfside and Cullendulla Beaches.

To help assess the likely consequences of constructing a physical barrier along the Wharf Road foreshore WBM (2000) examined historical shoreline positions in the Wharf Road area between 1898 and 1999. The assessment procedure was based on four hydrographic charts (1898, 1922, 1931 and 1978) and eight aerial photographs (1949, 1964, 1977, 1981, 1982, 1986, 1987, and 1999). Based on this assessment WBM estimated the percentage location of the high tide boundary and how often a range of different wall profiles would be exposed to direct flows. This was used as an indicator of the time a beach would be located in front of various wall alignments.

The above (WBM, 2000) assessment procedure has a number of inadequacies. One particular problem is the heavy (50%) weighting given to the ten year period between 1977 and 1987 when the foreshore alignment shape was wave dominated and not typical of past conditions. Another is the use of four hydrographic charts which given these a one third weighting. These charts potentially introduce substantial errors because the charts are predominantly indicators of channel depths for navigation. The foreshore information was usually not physically surveyed and was generally added for visual reference only.

To address some of these problems the percentage location of the foreshore was recalculated with a weighting based on the number of years between assessment dates (see Appendix D). This was easily established from the progressive change in foreshore shape over time. Similar dates were used to the WBM (2000) assessment but without the 1978 hydrographic chart and with additional reference to 1962, 1980, 1984 and 1996 aerial photography. Three different methods were adopted:

- **Method 1** used a weighting based on the time periods between assessment dates,
- **Method 2** used a weighting based on half the time period for hydrographic charts to reflect their uncertainty,
- **Method 3** also used half for the hydrographic charts, but the weighting was double for the period since the bridge and the existing Wharf Road seawall were constructed to reflect the changed hydrodynamic conditions.

Diagram 4: Wharf Road Foreshore and Historical Alignments

This revised assessment procedure produced significantly different percentage alignments to those produced by WBM (2000) (see Appendix D).

Based on the available information, a constructed physical barrier on the landward side of the 100% line is unlikely to be exposed and so should have minimal adverse impacts on sediment movement (see Diagram 4). A barrier along the 40%, 60% or even the 80% line is likely to be exposed at some stage, and so will have some impact on sediment movement. The nature and extent of these impacts is very difficult to evaluate.

WBM (2000) have assessed that the impacts of a barrier along the eastern Wharf Road foreshore will be small and that even when theoretically exposed the barrier could have a sand beach which would allow access along the foreshore. This assessment may be valid, however, it is also possible that a major flood event could expose the wall and the increase wave reflections might maintain the wall in an exposed condition. Another scenario might be that a deep efficient scour channel could form along the fixed foreshore which would be more permanent than existing/past channels.

Such a channel could also increase the time in which there is no beach along the Wharf Road foreshore. Such a channel may also increase or decrease the volume of sand delivered to Surfside Beach, which may in turn result in sand moving from the beach on to and along the Wharf Road foreshore.

Because of the complexity of the shoals and the variability of the wave and current climate in Batemans Bay it will not be possible, even with extensive numerical or physical modelling to determine which foreshore scenario will develop. In keeping with the precautionary principle of ESD, a foreshore without a constructed barrier will definitely have a permanent natural beach foreshore and a wall landward of the 100% foreshore alignment should have a minimal probability of adversely impacting on coastal processes.

6.3 Options Assessment

There are several options and combinations of options available for the Wharf Road area including:

- Do Nothing.
- Environmental Planning:
  - buffer zones,
  - restrictive zonings.
  - voluntary purchase.
- Development Controls:
  - building setbacks,
  - minimum floor levels,
  - building protection (raising floors & hazard proofing),
  - relocatable assets.
- Protective Works:
  - training walls (raising and extending),
  - beach nourishment and training wall (raising).

6.3.1 Do Nothing

The Wharf Road area would continue to be at risk from wave impacts and ocean inundation if the "do nothing" option was adopted. Caravans and low lying residences, particularly those in the immediate foreshore area, could be seriously damaged by waves, and low lying areas would be inundated by over 1.0 m. Sand accretion on the foreshore could continue, although on past evidence this will be largely lost during the next major flood event. The existing car tyre groynes along the foreshore will not prevent future flood erosion.

The main advantage of a "do nothing" approach is that it involves no cost to implement. However, there are potentially very large direct and indirect costs associated with future inundation and wave impact damage in the area. Based on the work undertaken for the CBD/residential area, wave impact and inundation will cause damages in excess of one million dollars during a 1% AEP event, with an AAD of around \$60 000 and a present value of around \$850 000 at 7% pa over a 50 year planning period.

6.3.2 Buffer Zones/Restrictive Zonings

The western rock wall section of the Wharf Road area is zoned a combination of Open Space Recreation and low medium density Residential which has facilitated the development of caravan parks and residential dwellings. To help minimise future damage a restrictive zoning could be used which reflects the coastal hazards. However because the area is already highly developed, development controls such as minimum floor levels or building hazard proofing requirements will provide more effective methods of minimising future damage (see Section 6.3.5 and 6.3.6).

The eastern beach section is zoned for tourist/residential development up to the road reserve but with special conditions requiring new development to recognise and address the inundation and wave impact hazards in this area.

6.3.3 Voluntary Purchase

The developments most affected by a major storm/ocean inundation event would be the caravans and mobile homes sited along the immediate foreshore of much of the western area. However, these developments are crucial to the economic viability of the caravan park. As a result, any voluntary purchase program for this area would probably involve closure of the foreshore caravan park (the land is actually Crown Land leased to the park operators).

The total cost of such actions would include, not only the park closure price, but also the social and economic impacts associated with the loss of a major local tourist development. The resulting costs are difficult to assess, but are likely to be at least equivalent to the damages estimated for the "do nothing" option.

Voluntary purchase could also be used for the eastern beach area of Wharf Road where there is existing low lying and submerged private land. There are currently proposals/suggestions for development of both the land above and the land below high tide level. One of the key actions promoted by the NSW Coastal Policy 1997 is that "beaches and frontal dunes will be protected and only minor development will be permitted for essential public purposes". Strategic Action 1.4.5 requires that development proposals on the coastline or offshore which are threatened by coastal hazards, or where they pose a threat to the physical well being of the coastline, will be approved subject to conditions which minimise impacts, or rejected where they pose an unacceptable threat to the physical wellbeing of the coastline.

If development is approved for the low lying/submerged areas it will need to consider the coastal hazards and demonstrate that it does not adversely impact on the coastline in accordance with the Coastal Policy. Based on the assessment of coastal hazards in the preceding sections, it is highly unlikely that all the privately owned land would be approved for development because of the potential for such development to adversely impact coastal processes and to be adversely impacted by coastal processes.

As a result, voluntary purchase of some of the properties in this area may be a viable option. For the purposes of this Management Plan the 100% historical foreshore alignment has been adopted (see Diagram 4). Use of the 100% alignment is in keeping with the principles of ecologically sustainable development.

Adopting the 100% line, and with appropriate allowance for a foreshore seawall/dune development and access requirements could mean that, there may be an area along the eastern beach foreshore where development could occur. The nature, extent and value of this development will depend on negotiations currently underway between Council, DLWC and the landowners/developers. Land seaward of the 100% line should be returned to public ownership as part of the development approval, possibly as part of a foreshore land rationalisation scheme.

6.3.4 Building Setbacks

A building setback could be used to prevent (inappropriate) development in the immediate foreshore wave impact zone. For the western rock wall section, any such zone would need to be around 30 to 50 m wide and therefore take up a significant proportion of the caravan park area. As a result, this approach would seriously impact on the economic viability of the business and hence is unlikely to provide a feasible solution (as discussed above).

A building setback approach is also unlikely to be suitable for the eastern beach section. Because of the low lying nature of the land in this area any setback which reflected the inundation and wave impact hazards would necessarily include all the land between the foreshore and McLeod Street (unless it were raised). Such an approach would prevent development of the area.

6.3.5 Minimum Floor Levels

Council has a policy for new development and major redevelopment in flood liable areas (Appendix C) which sets a minimum residential floor level of 3.2 mAHD for the Wharf Road area. This level is based on the Ocean Inundation Study (PWD, 1989) and includes a 0.5 m freeboard. The policy has been applied to new development since 1988.

Council also has a policy for caravan parks implementable at changes of ownership or use (Appendix C) which reflects different risk considerations for commercial and private property. For Wharf Road this policy requires minimum floor levels for:

- Permanent/long term cabins - > 3.2 mAHD (1% AEP plus 0.5 m),
- Park owned vans & annexes - > 0.6 & 0.1 m above ground level,
- Private cabins/mobile homes - > 3.2 mAHD (1% AEP plus 0.5 m),
- Private towable vans & annexes - > 0.6 & 0.1 m above ground level,
- Tents - No minimum level.

In addition, the policy requires no permanent/long term sites in recession zones, and that park vans/cabins in the hazard zone must be relocatable.

Based on the 1% AEP ocean inundation water level determined for this study, the existing interim flooding policy would provide a freeboard allowance of around 0.8 m for permanent residential development near the foreshore (not including localised wave runup). This level should be adequate to prevent inundation damage provided measures are taken to protect against wave runup and impact.

The implementation of these policies has/will reduced the potential level of damages during a major overtopping event. However, there remain a substantial number of residential properties built before the policy started with floor levels well below the set standard. Further, there are a number of caravan park owned dwellings which are designed as moveable cabins, but which would be very difficult to raise or relocate during a major storm event.

6.3.6 Building Protection

As a complementary measure to the minimum floor level policy, Council could establish development control standards which require proposed developments to be designed to withstand the assessed coastal hazards. This option has the advantage of focussing on individual at-risk developments and providing protection where it is needed. The primary measure would involve having floors above inundation levels, but may also involve special foundation requirements, wall strengthening, wave runup barriers, flood proofing, etc.

The policy would target new development and major redevelopment, but also help address problems for existing development through a program of house raising and hazard proofing. Because of the extent of low lying land and the existing type of development it will be difficult to raise and hazard proof all the residential land in the western Wharf Road area. Further, protecting the caravan park and the eastern land area will effectively require raising the ground level. This in turn will involve reinforcing the foreshore to protect the fill from wave attack.

The total cost of implementing this option is difficult to assess without better defining the various design alternatives, but a cost in excess of \$2.5 million would appear likely given the extent of the works required.

6.3.7 Relocatable Assets

Theoretically it would be possible to have a mix of fixed structures such as administration and amenity buildings above the minimum floor level and relocatable assets such as mobile vans and cabins elsewhere. However, because the likelihood of significant flood level inundation is low (less than once in 20 years) it is highly likely that mobile developments will become at least semi-permanent through the construction of annexes, steps, shade structures, etc.

In the event of a major storm and flooding, little confirmable notification would be available prior to the storm occurring. As a result, relocation of the assets would need to occur while the storm was in progress, during high winds and rain, over flooded roads, probably at night. There would need to be an identified accessible safe relocation area which did not cause obstructions to the road and endanger other residences.

Because of the difficulties in implementing this option it is not considered viable.

6.3.8 Training Walls

To prevent wave overtopping inundation and wave impact damage to foreshore development and low lying areas in the western section, a physical barrier could be constructed. The most effective method would probably involve reconstructing the existing rock wall to a sufficient height (3.3 mAHD at least) to protect existing development (see Figure 4a). The raised wall may detract from the amenity of the caravan park area.

To ensure that the wall will perform adequately, it should consist of two layers of armour stone (1.0 m in size) with a toe depth below zero mAHD. Raising McLeod Street to tie into high ground and appropriate drainage and leaching controls, including a layer of geotextile fabric, would be a necessary part of this option. To reconstruct the existing wall plus raise McLeod Street would require some 4 000 tonnes of rock and cost approximately \$500 000.

To provide protection to at least part of the existing unzoned private land along the eastern beach section a wall could be extended some 400 m along the foreshore. Varying the alignment allows either the number of lots to be maximised or the length of time the wall might have a sand beach on its seaward side to be maximised.

As discussed in Section 6.2.2, it is not possible to accurately predict the effects of constructing a wall on sediment movement in Batemans Bay. The two most likely significant effects are:

- periodic loss of a sandy beach along the Wharf Road foreshore,
- changes to sediment supply rates from the channel margin shoal to the nearshore of Surfside and Cullendulla Beaches.

For any alignment seaward of the historical 100% foreshore alignment it is highly likely that there will be some periods when there is no beach along the wall. The further out the wall, the less likely there will be a beach. This problem is exacerbated by the fact that once the seawall is exposed, increased wave reflections would act to maintain a no beach condition longer than would occur without the wall.

The assessment of foreshore locations undertaken for this study (see Appendix D) shows that the distance between the 100% and the 60% alignments is not great. This result suggests that the potential error margin is small, and that the historical 60% line could easily become the future 100% line (or vice versa), particularly given the irregular foreshore shape.

The problem with sediment resupply to Surfside and Cullendulla Beaches is even more complex. While any wall alignment will impound sand and potentially remove it from the flood/non-flood system, this is unlikely to be a significant problem unless the wall extends well into the channel margin shoal (WBM, 2000).

The more significant and difficult to assess problem is the effect the wall will have on the secondary channel along the Wharf Road foreshore, and how this will impact on both flood scour and the subsequent movement of sand from the channel margin shoal on to the beaches. Any change which reduces sediment supply to beach nearshore zones could result in ongoing erosion/recession.

To construct a seawall to around 3.3 mAHD along a beach alignment over a distance of around 400 m would require approximately 10 000 tonnes of rock and 4 000 m^3 of fill plus drainage works etc. (see Figure 4a). As a result the total cost would be around $1 400 000.

6.3.9 Beach Nourishment and Training Wall

Rather than using rock, an artificial beach dune could be used to protect the unzoned beach (eastern) section of Wharf Road. This would be undertaken in combination with raising the existing training wall (western) section (as above). The advantage of this option is that there would always be a sandy beach along the Wharf Road foreshore. The disadvantage is that the artificial dune may be eroded away during a major flood event, which would seriously threaten any development behind the dune.

For this option, the existing wall would be raised to around 3.3 mAHD over 500 m and the beach dune to at least 3.5 mAHD over some 400 m long (see Figure 4a). Approximately 4000 tonnes of rock and 16 000 m^3 of sand would be require, with a total cost of around $700 000 if the sand was sourced locally (from the channel margin shoal or Cullendulla Creek entrance shoal) or around $1 100 000 if the sand was imported.

6.4 Options Summary

As discussed above, the main coastal hazards for both the western rockwall and the eastern beach sections of the Wharf Road area are coastal inundation and wave impacts as a result of wave overtopping of the foreshore. Sediment movement associated with a long term cycle of flood/non-flood foreshore erosion and accretion is also a problem. The assessment of management options found (see Summary Table 4 and Figures 4 and 4a):

- **Do Nothing** - the present value of the likely inundation damages would be around $850 000 over a 50 year planning period. No value was assigned to sediment movement damages.

- **Environmental Planning** - buffer zones and restrictive zonings were not considered to be effective methods of minimising damage. Voluntary purchase may be feasible for both the western caravan park development and for the eastern beach area. Some land ownership rationalisation may be desirable along the eastern foreshore.

- **Development Controls** - the existing minimum floor level freeboard (0.8 m) is adequate to prevent inundation and should help minimise any wave runup impacts. Controls on permanent caravan park developments near the foreshore should also assist. The cost of implementing a program of building protection, such as house raising and land raising, was estimated to exceed $2.5 million.

- **Protective Works** - raising the existing foreshore training wall to protect development was estimated at around $500 000, but may have negative amenity impacts on the caravan park. The provision of a rock wall or sand dune to also protect the eastern foreshore area was estimated at $1.4 and $1.1 million respectively. Depending on the alignment, a fixed wall option could result in wider sediment movement problems.

Based on the above, continuing with the existing floor level development controls would be advisable, but the associated building protection, particularly large scale landfills, is unlikely to be feasible. Upgrading and raising the rock wall along the western section of Wharf Road would appear to be an economically feasible option, but with some negative social/user impacts. Voluntary purchase may be suitable for both sections, possibly with some land ownership rationalisation along the eastern foreshore.

7. SURFSIDE BEACH

7.1 Coastal Hazard Review

Surfside Beach is the innermost beach of the Northern Zone (see Figure 1). Surfside Beach is potentially subject to *coastal inundation* due to wave overtopping, as well as *beach erosion* hazards (particularly along the eastern foreshore) and *shoreline recession* mainly linked to climate changes.

The beach itself is some 900 m long, and spans between Hawks Nest and the rocky point near Timbara Crescent/McLeod Street (see Figure 5). The beach dune height is a consistent 3 mAHD along the eastern portion following beach nourishment in early 1997, but drops to around 2.5 mAHD at the western end. An access pathway and stormwater pipe are sited through the dune in the eastern corner of the beach.

Behind the dune the land has been levelled at around 2.3 mAHD for a residential subdivision. The residential development includes a row of some 35 houses (along Myamba Parade) backing on to the dune for the full length of the beach. Several property boundaries in the eastern corner of the beach are within 20 m of the erosion scarp, and one residence is 25 m from the scarp.

The coastal processes operating at Surfside Beach were examined in the Vulnerability Study (CRMD, 1996), in a study specifically of beach erosion (PBP, 1997), and in the Estuary Processes Study (WBM, 1999). An earlier assessment of flooding in the back beach area (PBP, 1992) also provides relevant information on dune overtopping. These studies were reviewed as part of this Management Plan and an assessment made of the likely coastal hazards.

7.1.1 Coastal Inundation

The Vulnerability Study estimated foreshore setup levels of 2.8 mAHD along Surfside Beach for the 1% AEP event. This level includes around 1.0 m for wave setup, assuming a 1.5 m nearshore significant wave height. It also includes an allowance of 0.2 m for uncertainty. The Vulnerability Study also estimated 1.4 m (western) and 1.0 m (eastern) wave runup allowances, assuming theoretical extended dune profiles.

Based on the available information, the estimated levels (not including uncertainty) appear to be reasonable. Discounting for the uncertainty factor (0.2 m), but including the 50 year climate change factor (0.2 m) does not change the estimated level. The foreshore setup level (with wave setup) is therefore approximately at the same level as the dune crest, and wave overtopping would occur along the beach during major storm events. Wave overtopping would be worse in the western portion of the beach where the dune level is lower.

The rate of wave overtopping was calculated for an assessment of flooding at Surfside Beach (PBP, 1992). This assessment was based on the findings of the Batemans Bay Oceanic Inundation Study (PWD, 1989) and is consistent with the Vulnerability Study. The flooding assessment calculated a 1% AEP dune overtopping rate of 40 m^3/s and then doubled this to 80 m^3/s as the best estimate for modelling purposes. This rate agrees with values calculated using the method set out in the Shore Protection Manual (CERC, 1984), which gives an overtopping rate of around 0.23 m^3/s per metre length of beach at 2.5 mAHD height, or around 100 m^3/s.

7.1.2 Shoreline Recession

Comparison of June 1942 and April 1993 aerial photography undertaken for the Vulnerability Study showed that over the 51 year period there has been no shoreline recession over most of the beach. Over the western two-thirds there appears to have been accretion of around 0.4 m^3 per metre length of beach above zero AHD on average each year.

The observed changes to the beach profile appear to be associated with the flood/non-flood cycle which affects the resupply condition of the nearshore beach zone off Surfside and Cullendulla Beaches. As outlined in Section 2.4, scour through the channel margin shoal by major floods deposits sand into the region between Surfside Beach and Square Head. This sand is then reworked back onto the channel margin shoal and the beaches.

The supply of sand to Surfside Beach under this process tends to be periodic. Under some conditions large quantities of sand are moved on to the western part of the beach by waves (WBM, 1999). This condition potentially represents the start of a wave dominated shoreline condition in the Wharf Road area (see Section 6.1.3) and based on historical aerial photographs and hydrographic chart data this occurs quite infrequently.

However, a more common method of sand resupply to the western part of Surfside Beach also exists during nearshore current dominated conditions. The presence of a secondary tidal channel along the Wharf Road foreshore (see Diagram 4) distributes sand from the channel margin shoal both towards Surfside Beach and into the main channel depending on the prevailing current. Sand directed towards Surfside Beach can then be worked on to the western part of the beach by wave action.

The volume of sand supplied to the beach is dependent on the location and size of the secondary tide channel and input rates from the channel margin shoal. This sand is then moved east or west along the beach depending on shoal patterns in the bay and hence the refracted wave climate at the beach. It may also be related to the occurrence or otherwise of storms from the northern or southern sector.

Following extended periods of negligible or minor sand supply to the beach, wave induced storm currents scour the nearshore zone. As suggested in the Estuary Processes Study (WBM, 1999), depletion of the nearshore area increases wave energy and hence alongshore sand transport on the beach. This then increases the potential for storm erosion.

Based on the above it was assessed that Surfside Beach is not undergoing long term recession, but is subject to substantial medium/long term movements depending on the flood/non-flood cycle. Erosion along the eastern part of Surfside Beach and Cullendulla Beach over recent years is part of this process.

The contribution to shoreline recession from a medium Greenhouse sea level rise of 0.2 m over the next 50 years was calculated in the Vulnerability Study to be around 5 m. As discussed in Chapter 2, this is probably conservative (an overestimate of recession) because there is considerable bed sediment movement and supply associated with estuary processes (tidal and fluvial flows) as well as coastal wave/littoral processes (see Diagram 2). However, given the uncertainties, the recession rate is considered to be appropriate.

7.1.3 Beach Erosion

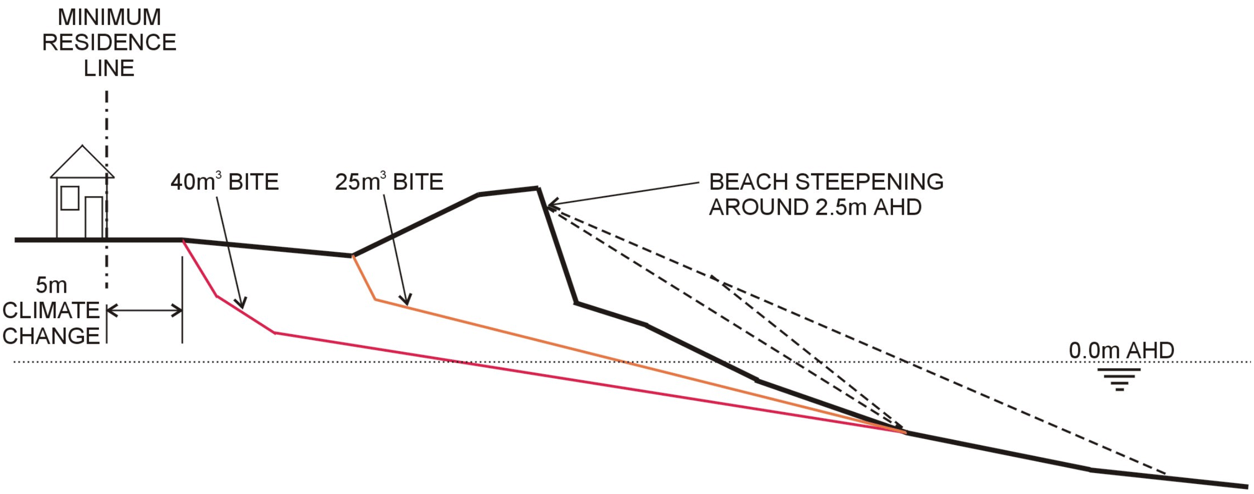
Analysis of aerial photography and beach profiles undertaken for the Vulnerability Study determined a beach erosion hazard level of 25 m^3 per metre length of beach above zero AHD. This was based on a comparison of aerial photographs from June 1942 with December 1959, and May 1972 with April 1990. Comparison of beach survey profiles, particularly those of July 1996 and June 1997, identified a loss of 37 m^3/m of sand from along the restored eastern portion of the beach. This was explained as a local variation due to the beach nourishment works undertaken in early 1997 (PBP, 1997).

Re-assessment of the available data, including August 1999 observations by Council, indicates that beach volume changes are largely related to periodic steepening or flattening of the beach profile from a line along the back of the dune at around 2.5 mAHD with little movement beyond that line (see Diagram 5). These changes are probably related to the flood/non-flood shoal cycles, and steepening of the beach profile as a result of the current non-flood conditions and depletion in the nearshore zone. It may also be related to a predominance of north easterly wave climate conditions over recent years.

Based on the above, it is reasonable to assume that erosion along the eastern portion of the beach could continue until the underlying bed/wave conditions in the bay change. This could be many years. Further, the beach slope in the eastern portion is now around 1:12, the steepest in 56 years. Therefore, it is possible that the beach will not steepen further around the historic 2.5 m dune line, but will instead erode past that line.

Assuming this possible worst case, individual storm bites of up to 25 m^3/m or a series of storm bites of up to 40 m^3/m in total are probable in the medium term. Erosion of the beach dune by around 10 m during one major event or by 20 m in a series of events is therefore considered possible.

Diagram 5: Surfside Beach Steepening and Erosion/Recession Hazard



7.2 Coastal Hazard Impacts

7.2.1 Coastal Inundation

In relation to wave overtopping and coastal inundation, the flood assessment (PBP, 1992) identified wave inundation levels of between 2.3 and 2.8 mAHD along Myamba Parade immediately behind the dune. Flow velocities as high as 1.5 m/s were estimated to occur between the properties at the western end. Beyond Myamba Parade water levels dropped to around 2.2 mAHD and flow velocities to less than 0.25 m/s.

Based on this assessment, damage to the beach properties along Myamba Parade as a result of wave overtopping and inundation during a major/design storm event would be extensive.

7.2.2 Coastline Recession and Beach Erosion

From the hazard review, coastline recession of up to 5 m as a result of climate changes plus a series of storm bites without recovery totalling up to 20 m are likely within the next 50 years along Surfside Beach. The back beach erosion escarpment is currently more than 15 m from the nearest private property boundary, and 25 m a from residential building. Recent erosion has already damaged the beach access path and stormwater drain in the eastern corner of the beach.

If the current non-flood depleted nearshore condition continues, and if there is a series of severe storms without adequate beach recovery, erosion and climate change induced recession are likely to become significant problems which would physically threaten substantial private development along the beach within a 50 year planning period.

7.3 Options Assessment

As a result of the identified coastal hazards, an investigation into “a contingency plan for the Surfside Beach area for those properties whose boundaries are threatened by ocean inundation” was undertaken (PBP, 1997). The study addressed the problems of beach recession and storm erosion, although wave overtopping and the resulting inundation was identified as the key coastal hazard, but was not addressed

The study found that short term actions were neither feasible nor necessary for beach recession and erosion. Four long term management strategies were examined, but ongoing monitoring of the problem was recommended. The four strategies were:

- a seawall or terminal revetment,
- beach nourishment,
- groynes and beach nourishment,
- do nothing.

As part of this Coastline Hazard Management investigation, a range of options have been examined which address both the wave overtopping/inundation hazard, as well as the long term recession/storm erosion hazard. The options considered include:

- Do Nothing,
- Environmental Planning:
    - voluntary purchase,
- Development Controls:
    - minimum floor levels,
    - building protection,
- Dune Management,
- Protective Works:
    - beach nourishment,
    - back beach drainage,
    - seawalls,
    - groynes.

In addition to these options it would be possible to combine some options, such as dune nourishment along the western part of the beach, and a revetment wall or individual property protection along the eastern part.

7.3.1 Do Nothing

The major consequence of adopting the “do nothing” option is the risk of inundation damage to properties along Myamba Parade, especially at the western end, and foreshore erosion and property damage along the eastern end. The number of houses affected by inundation would be between 20 and 30, depending on the ocean level and wave climate. Three properties may be affected by foreshore erosion, one or two houses may be threatened.

The average damages for a residential property inundated by 150 mm water is approximately $10 000. On this basis, the AAD for inundation would be around $14 000 with a net present value of $200 000 based on a discount rate of 7% over a 50 year planning period.

In relation to foreshore erosion, assuming a worst case erosion scenario (the loss of a beach front home and damage to two others within ten years) the cost would be around $400 000 and have a present value of around $250 000. A more realistic time frame, say the next 20 years, would reduce the present value to less than $150 000.

In total the combined present value cost of both inundation and foreshore erosion damages would be around $400 000.

7.3.2 Voluntary Purchase

The properties on Myamba Parade along the beach foreshore are mainly substantial developments with immediate beach access. As such they would command a high purchase price. Based on recent sales, an average of around $400 000 each could be expected. Clearly a voluntary purchase program would exceed several million dollars and be many times the assessed damages.

7.3.3 Minimum Floor Levels

Council's interim coastal flooding standard for the area is based on the 1% AEP ocean inundation level, plus a 5% AEP river flood allowance, and 0.5 m freeboard (ESC, 1997). This standard sets a minimum residential floor level of 3.2 mAHD over most of the Surfside Beach area based on the Ocean Inundation Study (PWD, 1989), and is consistent with the findings of this study.

Implementation of the policy has reduced the potential for damage during a major overtopping event. However, there remain a substantial number of properties built before the policy started with floor levels below the set standard. Further, this option does not address the potential problem of damage to properties as a result of wave impacts and local scour. Additional foundation support or scour prevention measures could be required on some properties.

The actual requirements would need to be assessed on a development specific basis. Possible options could include buried foundations say >0.5 m below surface level, or subsurface retaining /deflection structures such as rock or concrete walls, or geotextile sausage barriers.

7.3.4 Building Protection

This option involves identifying at-risk properties and reducing potential damages by implementing site specific measures such as deflection walls, levees and inundation proofing. Although expenses would be incurred at the individual properties, cost savings may accrue as a result of not providing unnecessary protection.

The individual nature of this option means that implementation costs are highly variable but an indicative cost of around $15 000 may be required to effectively seal a house against the ingress of water and $80 000 to provide a barrier/protection against property loss associated with storm erosion. Assuming 20 properties affected by inundation and three by erosion, the total cost of this option would be around $550 000. Deferring erosion protection works until the problems became more immediate would substantially reduce the present value costs to around $300 000.

7.3.5 Dune Management/Beach Nourishment

This option involves raising the crest of the beach dune to at least 3.6 mAHD to prevent wave overtopping in all but major storm events, and resupplying sand to the eastern corner as a buffer against future storm erosion and recession. The advantage of this option is that it provides a physical barrier to coastal inundation but does not have a major visual impact on the environment, although ocean views from some residences would be affected.

Currently, the dune height at the western end is around 2.5 mAHD and would need to be raised by over a metre in places (see Figure 5). As an initial stage, minimal work could be undertaken to raise the dune to above 3.6 mAHD, but with limited nourishment. This would substantially reduce the risk of dune overtopping and back beach inundation. With some local reshaping plus the importation of 6000 m^3 of sand, the cost of the works including fencing and revegetation would be approximately $200 000. If required, this work could include contingency planning for temporary storm bite protection.

To provide a buffer against future storm erosion, substantial nourishment by sand at around 40 m^3/m would be required in the eastern corner. In total approximately 20 000 m^3 could be required over a 50 year period, which would then need to be fenced and vegetated. The estimated cost of this additional work would be in the order of $550 000. Note, this does not include resupply of the depleted nearshore zone which could require in the order of 200 m^3/m (WBM, 1999).

Substantial beach nourishment could be delayed until the erosion scarp was within 15 m (that is approaching one storm bite) of a residential building. The work could then be undertaken in smaller stages of say 5000 m^3 of sand, with the worst areas addressed in response to erosion/runup events. The main disadvantage is that the work and associated disruption would need to be repeated several times within the 50 year planning period.

Based on initial overtopping works and then staged nourishment repeated at say 10 to 15 year intervals, the cost of each additional stage would be around $200 000 (>$750 000 in total over 50 years). The present value cost using a 7% discount rate would be around $400 000.

7.3.6 Seawalls

Another option to prevent coastal inundation and beach erosion would be to build a revetment seawall to a satisfactory standard and height. A minimum crest level of over 4.5 mAHD would be required to prevent theoretical wave overtopping. An 800 m long revetment wall, consisting of two layers of armour stone (around 1.0 m in size), including a layer of geotextile fabric, with a crest of 4.0 mAHD and a toe apron would prevent back beach erosion and all but limited overtopping during major events.

In total some 22 000 tonnes of rock would be required for the full wall, plus appropriate drainage and leaching controls, including a layer of geotextile fabric (see Figure 5). The cost of such an option would be around $1 600 000.

A major drawback of the option (apart from cost) would be the loss of a sandy beach, and a reduction in visual appeal and recreational use, especially in light of the fact that the beach fronts an area of residential development. To reduce this impact initially, excavated sand could be placed over the wall and stabilised with dune vegetation. This would increase costs to around $1 800 000.

A shorter wall could be used to protect the eastern section of the beach, but overtopping could still occur along the western section, and exposure of the wall by storm erosion would cause the sandy beach amenity to be lost. The local wave climate would also increase possibly resulting in erosion of the unprotected part of the beach. A shorter wall would cost proportionally less, around $450 000 for a 200 m wall.

7.3.7 Back Beach Drainage

As inundation levels along Myamba Parade are between 2.3 and 2.8 mAHD, properties may experience up to 0.5 m of ponding. To reduce damages from overtopping of the dune, drainage paths could be provided in the back beach area, discharging to the natural watercourse leading from the wetland on the western side of the beach. This option would not address the erosion problem, nor mitigate damages to the properties on the beach dune.

A meaningful estimate of likely costs for a back beach drainage system can only be achieved with more detailed investigation, but excluding the need to provide a drainage easement, any such scheme is likely to require at least 600 m of major drainage works and cost several hundred thousand dollars. Measures to protect the beach front properties would still be required.

7.3.8 Groynes/Breakwaters

Groynes along the beach or a single breakwater off Hawks Nest have been suggested as possible options. Such structures could be used to modify wave patterns on the beach and help prevent ongoing beach erosion. As discussed in Section 7.2.2 beach recession and erosion are significant problems but are unlikely to threaten residential buildings within a 50 year timeframe.

By diffracting waves away from the eroding sections of the beach the suggested structures could reduce wave erosion impacts in those areas. However, detailed investigations including modelling would be required to identify the likely impacts given a range of wave directions and bed sediment

profile conditions which can occur at the site. It is highly likely that induced changes to the existing wave patterns and sediment movement could also increase erosion at other locations.
A series of three 80 m beach groynes, or a 200 m breakwater off Hawks Nest is likely to require at least 15 000 t of rock and cost in excess of $1 500 000.

7.4 Options Summary

The coastal hazards for the Surfside Beach area are coastal inundation, mainly wave runup and overtopping between properties on the beach dune, and a combination of coastline recession as a result of climate change and storm bite erosion. Storm bite erosion has not historically been a hazard but appears to have developed over recent years as a result of the depleted condition of the nearshore zone as part of the sand movement flood/non-flood cycle. The assessment of management options found (see also Summary Table 5 and Figures 5 and 5a):

- **Do Nothing** - the present value of coastal inundation and erosion damages would be around $400 000 over a fifty year planning period.

- **Environmental Planning** - options such as voluntary purchase are not feasible because of the extent and value of the existing development.

- **Development Controls** - existing minimum floor level freeboards are probably adequate (0.5 m) for back beach residential developments, but are not adequate to prevent wave runup damage along the foreshore dune. The cost of implementing a program of works in the short term to hazard proof existing development would be around $550 000, but could be significantly less (around $300 000) if erosion protection works are delayed until the problems become more immediate.

- **Dune Management/Protective Works** - beach reshaping and nourishment to raise the dune to prevent overtopping during major storms would cost around $200 000. This could include contingency work on temporary storm erosion protection. Substantial nourishment as a buffer against future storm erosion would cost around $550 000. Staged nourishment when necessary (erosion scarp within 15 m of a residential building) would cost around $200 000 every 10 to 15 years (total >$750 000 over 50 years) with a present value cost around $400 000.

- **Protective Works** - A seawall would provide a more permanent solution but would be more expensive (around $1 800 000 for a full beach wall), and would result in the sandy beach being lost. This would be both socially and economically undesirable. A shorter wall would be proportionally less (around $450 000 for a 200 m wall). Constructing back beach drainage would be very costly and would not fully address the hazards. Constructing groynes or a breakwater to diffract wave energy away from the beach would cost at least $1 500 000 and could adversely impact on other areas.

Based on the above, nourishing the beach would be economically feasible (and would probably be socially feasible) if minimal work was undertaken initially for overtopping plus some contingency erosion protection, and major work was delayed until necessary. A seawall resulting in loss of the sandy beach amenity is unlikely to be socially or economically feasible. Continuing the current policy of minimum floor levels with additional hazard proofing requirements in the immediate foreshore area would be economically feasible, particularly if erosion protection works were delayed until the problems became more immediate.

8. CULLENDULLA BEACH

8.1 Coastal Hazard Review

Cullendulla Beach (also known as east Surfside Beach) is separated from Surfside Beach by the small rocky promontory of Hawks Nest (see Figure 1). The coastal hazards along Cullendulla Beach are similar to those at Surfside Beach i.e. *beach erosion*, *shoreline recession* and *coastal inundation*, although an absence of coastal residential development makes the coastal hazard problems appear less acute.

The beach itself is some 800 m long and faces south (see Figures 1 and 6). At the eastern end there is a large sand shoal which has formed near the entrance to Cullendulla Creek, in the lee of Square Head. Along the western edge of that shoal there is a large seagrass bed.

The back beach area has a height of between 1.5 and 2.0 mAHD. The land backing the beach is part of the Cullendulla Creek wetland area which has recently been designated a National Park under the control of the NSW National Parks and Wildlife Service. An important sewer rising main, a telephone cable and a now disused access track run along the back of the beach.

Photogrammetric work undertaken for the Vulnerability Study (CRMD, 1996) provides an indication of storm erosion and ongoing recession. The Estuary Processes Study (WBM, 1999) discusses sediment movement in the area, and an earlier flooding assessment of the area (PBP, 1992) also provides some relevant information on catchment runoff and coastal inundation. These studies were reviewed as part of this study and further work was undertaken to quantify climate change, elevated ocean levels and wave overtopping issues.

8.1.1 Shoreline Recession

Comparison of the June 1942 and April 1993 aerial photographs undertaken for the Vulnerability Study showed that, over a 51 year period, there has been consistent ongoing scarp recession along Cullendulla Beach. The rate varies from 1.3 m/yr at Hawks Nest to 0.46 m/yr near the Cullendulla Creek shoal seagrass bed. East of the seagrass bed near the creek entrance, the recession rate was 1.76 m/yr. The June 1995 profiles showed that this movement was significant down to a level between -0.5 and -1.5 mAHD.

The contribution to shoreline recession from a medium climate change sea level rise of 0.2 m over the next 50 years was calculated in the Vulnerability Study to be around 6 m which given the uncertainties is considered to be appropriate.

8.1.2 Beach Erosion

The Vulnerability Study determined a beach erosion hazard of 20 m^3 per metre length of beach above zero AHD based on a comparison of the May 1972 and April 1990 aerial photography. This assessment was an approximate envelope/average for the beach, which for the period examined varies from 30 m^3 per metre length near Hawks Nest to accretion near the Cullendulla Creek shoal seagrass bed.

The assessment did not consider erosion below zero AHD, which based on the 1995 profiles would increase the storm erosion rate by around 50%. However, the assessment also made no allowance for the 18 years of ongoing shoreline recession which were incorporated into the assessment. Removing ongoing recession reduces the assessed storm bite to zero.

To provide an estimate of storm bite for this planning study, the equivalent of five years scarp recession was adopted as the storm bite allowance. This is equivalent to 20 m^3/m near the creek shoal.

8.1.3 Coastal Inundation

The Vulnerability Study estimated peak 1%, 2% and 5% AEP foreshore setup levels at 2.0, 1.9 and 1.8 mAHD respectively, including wave setup. Discounting the 0.2 m for uncertainty but including the 50 year Greenhouse sea level rise, these levels are at or slightly higher than, the back beach level of between 1.5 and 2.0 mAHD. Note that the estimated nearshore significant wave height is 1.5 m, and the resulting theoretical wave runup is 0.8 m.

Based on the above, significant wave overtopping of the back beach area along Cullendulla Beach could and does occur. The flooding assessment of the back beach area (PBP, 1992) was based on 40 m^3/s, but adopting the still water levels from the Vulnerability Study and using the method set out in the Shore Protection Manual (CERC, 1984), the overtopping rate for Cullendulla Beach would exceed 0.3 m^3/s per metre length of beach (or more than 250 m^3/s for a 1% AEP event).

8.2 Coastal Hazard Impacts

8.2.1 Coastline Recession and Beach Erosion

Cullendulla Beach is affected by beach erosion, long term recession and wave overtopping. Based on the above review, long term recession over a 50 year planning period would be around 70 m near Hawks Nest, falling to 28 m near the entrance shoal seagrass beds. Beach erosion due to “a 5 year storm bite” along the beach could be up to 15 m.

Storm erosion and long term recession of the beach in the Hawks Nest area appear to be related to wave induced erosion, probably associated with depletion of the nearshore zone by wave induced storm currents (see Diagram 2). This has resulted in foreshore/beach loss which is already threatening the access track and the remaining sclerophyll forest eucalypts along the beach, and could threaten the back beach rising main and the telephone cable within 10 to 15 years.

Recession (and storm erosion) from the seagrass beds east to the creek entrance appears to be associated with entrance channel movements rather than wave induced changes. Based on past movements, beach loss could exceed 90 m over 50 years, but significantly larger or smaller movements around the entrance are possible.

8.2.2 Coastal Inundation

Wave overtopping and inundation of the back dune area will occur during major storm events. Based on the above assessment, wave overtopping of the dune would cause large inflows of ocean water into the estuarine wetlands behind the beach. This water would then drain back to the ocean via Cullendulla Creek. A 1% AEP flood level of 2.2 mAHD was estimated for the back beach area. Scour damage and creek bank erosion in the back beach wetlands could be significant due to the large volumes of ocean water involved.

8.3 Options Assessment

The hazards review identified the access track, telephone cable and sewerage line which run along the back dune area of Cullendulla Beach as being under threat (see Figure 6). For this study a range of options have been examined which address both the wave overtopping/inundation hazard, as well as the long term recession/storm erosion hazard. The options considered include:

- Do Nothing,
- Protective Works:
    - relocate assets,
    - beach nourishment,
    - revetment walls,
    - groynes.

8.3.1 Do Nothing

Management of Cullendulla Beach as a National Park is the responsibility of the NSW National Parks and Wildlife Service, although responsibility for the sewer line remains with Council and Telstra owns and is responsible for the telephone cable. If a “do nothing” approach was adopted coastline recession, beach erosion and coastal inundation would continue to threaten the two items of infrastructure, and based on historical recession and erosion rates, they would probably need to be repaired/replaced within fifteen years.

The cost of a “do nothing” option, would therefore be the deferred costs to repair, protect or move the infrastructure. Based on the following Sections, this would be around $500 000 (see Section 8.3.2). The intangible costs incurred by the community and the direct costs to the environment from damage to and loss of the sewerage and telephone system would also be major (assumed >$500 000).

Assuming major damage does not occur for 7 years, the present day cost of this option would be over $600 000 based on a 7% discount factor.

8.3.2 Relocate Assets

In the absence of residential development this option involves relocating services away from the impacted zone. The sewer rising main conveys sewage from the Long Beach area and discharges it to a receiving gravity line in Myamba Parade. Some 1000 m of rising main could be relocated approximately 50 m north. The telephone line runs along a similar alignment to the rising main and could also be relocated to the north. The creek crossings would remain.

The cost of the sewer relocation would be in the order of $350 000 including access and approximately 1000 m of extra pipe. The cost of the cable relocation would be around $150 000. The environmental impacts of relocating these assets would need to be considered, but with appropriate restoration works could be minimal.

8.3.3 Beach Nourishment

Building up the beach dune to 2.5 mAHD would protect the three items of infrastructure from scour associated with significant wave overtopping and replenish sand lost by ongoing recession. Based on the existing dune crest height of between 1.5 and 2.0 mAHD, approximately 130 000 m^3 of sand would be required for the 800 m long beach over a 50 year period.

The estimated cost to carry out this option, provided the sand could be sourced from nearby (such as the Cullundulla Creek entrance shoal), is around $1 400 000, with a net present value of $500 000 based on nourishment works every 10 years and a 7% discount factor. However, the environmental costs of this work is probably high, making local sand supply infeasible. If the sand had to be imported on a commercial basis, which is highly probable given the large quantities involved, then the total cost would exceed $4 000 000, with a net present value of around $1 400 000.

8.3.4 Seawalls

A revetment seawall to a satisfactory standard and height, with a minimum crest level of 2.8 mAHD would be required to prevent significant wave overtopping during major storm events.

The cost of importing some 20 000 tonnes of rock, and constructing a 800 m long revetment wall, consisting of two layers of armour stone (around 1.5 tonnes in size) with a crest of 2.8 mAHD and a toe apron is approximately $2 000 000. Appropriate drainage and leaching controls, including a layer of geotextile fabric, would be a necessary part of this option. The major drawback of the option (apart from cost) is the reduction in visual appeal and recreation use, especially in light of the fact that the beach fronts an area of National Park.

8.3.5 Groynes

The beach and back beach infrastructure could be protected by the construction of groynes along the beach. This option would also include beach nourishment and dune reshaping to minimise wave overtopping.

Based on the construction of three groynes around 70 m in length, using 10 000 tonnes of rock, and raising the dune to 2.5 mAHD with 10 000 m^3 of sand, an approximate cost for this option would be in the order of $1 300 000. Note however, considerably more investigation into coastal processes and groyne design requirements would be required before a reasonable estimate could be made.

8.4 Options Summary

Cullendulla Beach is part of a National Park but coastline recession and beach erosion have the potential to threaten a sewer rising main, telephone cable and access track along the beach. Wave overtopping and inundation of the back beach wetland also occurs. The assessment of management options found (see Summary Table 6 and Figure 6 and 6a):

- **Do Nothing** - the present value of likely future infrastructure and environmental damage would be over $600 000.

- **Protective Works** - the cost of relocating the assets now would be around $500 000. Works to protect the infrastructure such as beach nourishment, or construction of a seawall or groynes would cost around $1.4 million, $2.0 million and $1.3 million respectively. Beach nourishment could cost around $500 000 if a local source (such as Cullendulla Creek shoal) could be utilised but this is unlikely because of the likely environmental impacts.

Based on the above, relocating the assets when required in the medium future would appear to be the preferred option. Nourishing the beach with locally sourced sand would be feasible if environmental impacts could be negated. Doing nothing until the infrastructure was damaged would be economically feasible but not socially or environmentally desirable or justifiable.

9. LONG BEACH

9.1 Coastal Hazard Review

Long Beach is the largest of the northern zone beaches (2.2 km) and lies between Square Head and Chain Bay (see Figure 1). A number of coastal hazards have been identified for Long Beach including:

- *beach erosion*,
- *shoreline recession*,
- *coastal inundation* due to wave overtopping.

The beach faces south/south-east (see Figures 1 and 7) and is exposed to semi ocean wave climate conditions, with an estimated significant inshore wave height during major storms of around 1.5 m (PWD, 1989). Offshore bay depths average around -10 mAHD, with a slope of around 1:500. The nearshore beach area commences at around -5.0 mAHD and has a slope of around 1:30.

The western third of the beach has a dune height of around 5 mAHD. The back beach area is largely undeveloped open sclerophyll forest. The central third of the beach also has a dune height of around 5 mAHD, except near the outlet to Reed Swamp lagoon (at Sandy Place, west of Long Beach Road). Reed Swamp backs the central part of the beach and the dune level near the outlet drops to around 3.0 m AHD. There are 16 residential properties along the older part of Sandy Place which back directly on to the dunes near the outlet.

The eastern third of the beach near Chain Bay has a further 30 properties facing the beach on the northern side of Bay Road (see Figure 7). The beach foreshore along part of Bay Road is protected by a rock revetment wall which was built in the 1980's. The wall is mainly buried and has an irregular crest level between 3 and 3.5 mAHD. The ground levels for most properties along the beach are above 3.5 m, although several lower properties and much of Bay Road are around 3.0 mAHD.

The Vulnerability Study (CRMD, 1996) includes an assessment of beach erosion above 1.0 mAHD between 1972 and 1990, and shoreline recession above 2.0 mAHD between 1942 and 1993. Additional investigations were undertaken for this Coastline Hazard Management Plan including an examination of:

- storm bite for the period between April 1993 and July 1997,
- dune scarp movements (long term and short term),
- long term recession rates (down to zero AHD),
- coastal processes and human interactions,
- climate change effects on recession rates.

9.1.1 Beach Erosion

The Vulnerability Study determined an erosion hazard volume (storm bite) above 1.0 mAHD of 35 m^3/m length along the western two-thirds of the beach from Square Head to the lagoon outlet, and 10 m^3/m length for the eastern third, based on comparison of the 1972 and 1990 photography. The actual calculated data showed the higher erosion rates along the beach adjacent to Sandy Place and the lagoon outlet. Volume changes outside this area were generally less than 20 m^3/m length, with rates around 10 m^3/m length along the western third (CRMD, 1996).

The eastern half of the beach, covering the area fronting Bay Road and Sandy Place was surveyed in July 1997 following the May storm. Examination of these profiles and comparison with the 1993 photogrammetric profiles shows that there was an average erosion bite of around 8 m^3/m length, with a maximum bite of 11 m^3/m length. Most of the loss was between 2.0 and 0.0 mAHD with some accretion evident in the photogrammetric profiles which went below the zero AHD level. There was no apparent change above the 2.0 mAHD level.

More detailed examination of the beach profiles between 1942, 1959, 1972, 1990 and 1993 showed that storm bite loss above 2.0 mAHD was generally matched by accretion between 2.0 and 0.0 mAHD (and lower). Erosion loss over the profile above 2.0 mAHD approached the adopted 35 m^3/m length value, only in the section of beach adjacent to Sandy Place between 1942 and 1959, and between 1972 and 1990. Also, only between 1942 and 1959 did the erosion loss above 0.0 mAHD significantly exceed 20 m^3/m length. Note that this period also coincided with both the development of residential allotments along Sandy Place as well as works to change and formalise the lagoon outlet location.

An examination of dune scarp movements between 1942 and 1993 shows that there has been:

- no significant scarp movement in the western third near Square Head,
- a loss of up to 20 m in the central third,
- a nett gain of less than 10 m in the eastern third along Bay Road.

The larger losses along the central third are probably related to lagoon entrance erosion and past outlet works. The net gain in the eastern third may be related to the rock revetment.

Examination of scarp movements between consecutive aerial photographs shows:

- maximum movements of 8 m in the western third between 1942 and 1959,
- up to 14 m in the central third between 1972 and 1990,
- 7 m in the eastern third between 1959 and 1972 (with up to 12 m at the middle drainage outlet)

Based on the above, and assuming scarp movements above 2.0 mAHD only (the zone of most movement), an erosion hazard volume/storm bite value of 20 m^3/m length (around 8 m storm bite above 2.0 mAHD) was adopted for the western third, increasing to 35 m^3/m length (between a 14 m and 20 m storm bite) for the central third, and dropping to 10 m^3/m length (up to a 10 m storm bite depending on the location of the revetment wall) for the eastern third (see Figure 7). These values represent an envelope of all the above information including total scarp movement over the last 56 years since 1942.

9.1.2 Shoreline Recession

The Vulnerability Study considered shoreline recession above 2 mAHD only. As mentioned above, a subsequent review of the beach profiles between 1942 and 1993 undertaken for this study clearly shows that changes above 2.0 to 2.5 mAHD (accretion and erosion) have been matched by changes below that level (erosion and accretion). Further, for all the profiles horizontal and vertical movements at around the 2.5 mAHD level have been negligible.

Along the western third of the beach, near Square Head, accretion between zero and 2.0 mAHD exceeds the losses above 2.5 mAHD by over 0.5 m^3/m/yr. As a result of the accretion, the inshore beach level has increased by about 1.0 m and the foreshore slope has flattened from around 1:16 to 1:20.

Along the central third of the beach adjacent to the lagoon outlet, accretion between zero and 2.0 mAHD has not exceeded the losses above 2.5 mAHD, resulting in nett losses of up to 1.0 m^3/m/yr. Accretion in the inshore area, however, has increased the bed level by up to 0.5 m and the foreshore slope has flattened from around 1:14 to 1:18.

Along the eastern third of the beach in the Bay Road/rock revetment area, there has generally been accretion above 2.0 m AHD, as well as between zero and 2.0 mAHD. The nett changes have not been great, but there has been an overall increase of around 0.2 m^3/m/yr. As a result of this accretion, the inshore beach level has remained at about the same level, but there has been a slight steepening of the foreshore slope from around 1:20 to 1:16.

The above assessment shows that since 1942 there has been no long term beach recession on Long Beach, but that there has been a change in beach slope with a flattening of the western two-thirds of the beach foreshore and a steepening of the eastern third. This change has occurred around the 2.5 mAHD level, resulting in some inland movement of the erosion scarp along the western part of the beach. However, this movement is reflected in the adopted erosion hazard value (Section 9.1.1) and is not long term recession.

The identified movements in the nearshore beach slopes are probably related to changed wave climate conditions along the beach. These changes may be related to:

- the flood/non-flood shoal conditions which are believed to affect the inner bay beaches (SKP, 1986),
- the construction of the Bay Road revetment,
- the predominance of north easterly wave climate conditions over recent years,
- previous dredging/training wall and disposal works in the bay,
- the result of periodic or long term climate effects.

Based on the above it is reasonable to assume that Long Beach will remain stable in relation to long term recession, other than for the effects of any future climate change sea level rises. Shoreline recession as a result of sea level rise was calculated for the Vulnerability Study using the Per Bruun rule (Bruun, 1981), and reassessed for this study. As a result, the predicted 8.0 m shoreline recession rate for the medium 50 year sea level rise was confirmed and adopted as the general shoreline recession for this study.

9.1.3 Coastal Inundation

The Vulnerability Study estimated a 1% AEP still water level of 2.8 mAHD along Long Beach. This level includes an allowance of 0.3 m for uncertainty, but does not include a 0.2 m climate change allowance. The level also includes around 1.0 m for wave setup assuming a 1.5 m nearshore significant wave height. The Vulnerability Study also adopted a 2.0 m (western), 1.5 m (central) and 1.2 m (eastern) wave runup allowance assuming theoretical extended dune profiles.

Based on the above, the foreshore setup level (without uncertainty but with a climate change allowance) plus wave runup is less than the dune height along the western third of the beach near Square Head (4.4 versus 5.0 mAHD), and for most of the central third (3.9 versus 5.0 mAHD). Only near the lagoon outlet and in the eastern third along Bay Road does the wave overtopping level exceed the dune height (3.6 versus 3.0 mAHD). Wave overtopping is therefore expected to occur near the lagoon outlet and along the eastern part of the beach near Bay Road during major storm events. Waves overtopping the dune would move up the lagoon outlet channel and wash over Bay Road, inundating low lying properties.

The effects of elevated ocean levels on back beach flooding was examined in the flooding assessment of the Reed Swamp lagoon (WP, 1991) but no allowance was made for wave overtopping. Using the procedure identified in the Shore Protection Manual (CERC, 1984), an estimated peak overtopping rate of around 5 m^3/s was determined for the design storm event at the lagoon outlet, with a further 15 m^3/s across Bay Road.

Flow entering the lagoon outlet would aggravate ponding in the lagoon area. The estimated 1% AEP lagoon flow from the flooding assessment (WP, 1991) was 8.5 m^3/s, resulting in a maximum lagoon level of around 3.5 mAHD. Foreshore setup levels of around 2.7 mAHD, wave runup to 3.9 mAHD and inflows of 5 m^3/s could raise lagoon flood levels, although the impacts are unlikely to be major.

The combined effect of high ocean levels and waves overtopping Bay Road would cause ponding in the low lying back beach strip along the road. Drainage of this area would be controlled by the ocean level and the flow capacity of the central stormwater drain. Visual assessment of the area indicates that a number of properties along Bay Road with floor levels at or around 3.5 mAHD could be affected. More extensive survey and numerical investigations beyond the scope of this planning study would be required to accurate determine inundation levels and impacts.

9.2 Coastal Hazard Impacts

9.2.1 Beach Erosion and Recession

Long Beach has been identified as potentially subject to beach erosion and climate change induced recession, as well as coastal inundation. An assessment of the available data undertaken for this study identified a process of ongoing beach scarp movements associated with a rotation of the beach slope around a horizontally and vertically fixed 2.5 mAHD point. This rotation mainly involved flattening of the foreshore beach slope along the western and central thirds of the beach and a steepening of the foreshore along the eastern third. The change in slopes has ranged between 1:14 and 1:20.

The adopted hazard lines for future 50 year potential scarp movement are shown on Figure 7. These lines are 8 m for the western third, between 14 and 20 m for the central third, and 10 m for the eastern third. In addition, coastal recession of up to 4 m as a result of climate changes over the next 50 years was also identified.

On the basis of these identified hazards, erosion or long term recession within a 50 year planning period could progress within residential property boundaries along the back beach area of Sandy Place and could become a problem for development adjacent to the lagoon outlet (see Figure 7). It is also possible that Bay Road could be damaged, or because of the revetment works, increased wave reflections could cause sand loss from that portion of the beach.

9.2.2 Coastal Inundation

The hazard assessment also identified wave overtopping of the beach dune as a potential problem, particularly in the eastern third. As a result of the modelling undertaken for a flooding assessment (WP, 1991), back beach inundation levels of up to 3.5 mAHD were identified along Sandy Place. The inclusion of wave overtopping effects could raise these levels slightly. There is also a potential for wave runup inundation levels of around 3.6 mAHD along Bay Road, depending on drainage conditions.

Based on this assessment, damage to some properties (around ten) along Sandy Place and Bay Road would occur as a result of wave overtopping and catchment runoff during a major design storm event. The extent of this damage has not been determined and is beyond the scope of this study.

9.3 Options Assessment

The Vulnerability Study (CRMD, 1996) identified a comprehensive list of options to deal with the identified beach erosion and ocean inundation problems. These options included:

- Do Nothing,
- Development Controls:
  - minimum floor levels,
  - building setbacks,
- Protective Works:
  - beach nourishment,
  - revetment walls.

Voluntary purchase has also been suggested as an option. However, as indicated previously, the beach erosion and coastal inundation hazards within a 50 year planning period are not substantial. Based on recent sales, the average value of a property would be around $350 000. The purchase of such properties is unlikely to be justifiable based on coastal hazards alone.

9.3.1 Do Nothing

The main consequence of a "do nothing" option is an ongoing ocean inundation risk to around 10 low lying properties around the Reed Swamp outlet and along Bay Road. Long term recession could also encroach upon a few private property boundaries along Sandy Place, and wave overtopping and the possibility of ongoing beach erosion could threaten the stability of Bay Road.

The average damages for a residential property subject to wave inundation up to 0.2 m is around $30 000. In the short term, an estimated AAD for the residential inundation hazard is around $15 000, or a net present value of $200 000 based on a discount rate of 7% over a 50 year planning period. An allowance of around $500/m or around $50 000 is also required for repairs to Bay Road after major storm events (5%AEP). This would bring the total present value of damages to around $300 000.

Additional damages associated with wave overtopping inflows into Reed Swamp could also occur to properties around the lagoon. An analysis of flooding around the lagoon would be required to determine these impacts.

9.3.2 Minimum Floor Levels/Building Protection

New development and redevelopment in the area can be protected from coastal inundation via the setting of minimum floor levels. However, existing properties that are under threat are not protected by this option. These properties could be made free of inundation damage by providing hazard proofing on a building by building basis.

Council already has an interim minimum development floor level for the coastal area of 3.2 mAHD, which is 0.5 m above the assessed maximum ocean foreshore setup level but less than the runup level of 3.6 mAHD. This minimum floor level should be adequate provided appropriate safeguards are implemented to prevent wave runup impacts on beach front properties, such as deflection walls and deep foundations.

The actual requirements would need to be assessed on a development specific basis. Possible options could include buried foundations say >0.5 m below surface level, or subsurface retaining /deflection structures such as rock or concrete walls, or geotextile sausage barriers. The cost of protecting beachfront properties from wave runup inundation at say $30 000 per property would be around $300 000.

For properties around Reed Swamp, a minimum floor level of 3.5 mAHD plus freeboard would be required to prevent catchment inundation damage. The extent and cost of protecting properties from catchment inundation/elevated ocean levels around Reed Swamp will need to be examined in more detail as part of a catchment management study.

9.3.3 Building Setbacks

Providing an erosion hazard setback for new development helps limit future damages. Council already has a development setback line based on work undertaken for the Vulnerability Study. The 50 year erosion hazard line (storm bite plus recession plus medium climate change ocean level rise) was re-assessed as part of this study (see Section 9.1.2 and Figure 7). To this an allowance of around 20 m should be added for the scarp slope and local variations. Existing affected properties are not protected under this option, but at this stage no homes or residences are threatened.

The only significant cost of the setback option is for implementation. A major disadvantage of this option is the reduction in the number of uses to which the affected land can be put.

9.3.4 Beach Nourishment

Raising the beach dune along the eastern third to above 3.5 mAHD will protect low lying properties and parts of Bay Road from inundation. Approximately 300 m of Bay Road is at or below 3 mAHD and several properties have floor levels in this range. The length of dune which requires building up is about 600 m, which translates to a volume of sand totalling approximately 3 000 m^3. Some additional drainage works would also be required. The estimated cost to carry out this option is $150 000 assuming the sand was imported on a commercial basis.

Raising the beach dune along the central third to above 3.5 mAHD would help protect low lying properties along Sandy Place and Reed Swamp. A further 1500 m^3 of sand plus additional works and flow control structures would be required at the entrance to Reed Swamp to prevent unwanted environmental impacts in this area. The cost of these works could exceed $200 000.

A disadvantage of these options is that substantial maintenance could be required after major storm events. Assuming the initial program of work was repeated every 10 to 15 years, the total expenditure would be around $1 000 000 in a 50 year period, or $450 000 for 10 000 m^3 of sand along the eastern section and $600 000 for a similar quantity of sand along the central section. The present value cost would be around $700 000 in total.

9.3.5 Seawalls

The existing seawall revetment fronting Bay Road is partially buried and has an irregular crest level. An option which would prevent coastal inundation damage to low lying properties along Bay Road and Sandy Place, and protect Bay Road itself, would be to rebuild the revetment wall to a satisfactory standard and height and extend the wall southwards past the Reed Swamp outlet with an appropriate flow control structure. A minimum crest level of 3.6 mAHD would be required to prevent wave overtopping.

The cost of constructing/reconstructing a 800 m long revetment wall using 24 000 tonnes of rock, consisting of two layers of armour stone (1.7 tonnes in size) with a crest of 3.8 mAHD and a toe apron is approximately $2 500 000. An appropriate Reed Swamp outlet structure, drainage and leaching controls, including a layer of geotextile fabric, would be a necessary part of this option.

A disadvantage of the option is the potential for it to increase steepening of the beach in this area and ultimately for the possible loss of the sand beach at the front of the wall.

9.4 Options Summary

The coastal hazards affecting Long Beach are beach erosion, climate change induced recession and coastal inundation. The problems are most severe around the entrance to Reed Swamp Lagoon. The assessment of management options found (see Summary Table 7 and Figures 7 and 7a):

- **Do Nothing** - the present value of damages to property and infrastructure (mainly roads) would be around $300 000 over a 50 year planning period.

- **Development Controls** - existing minimum floor level freeboards are adequate for most back beach areas but not for properties in the immediate foreshore zone subject to wave impacts where hazard proofing works are estimated at around $300 000. Additional building protection controls and building setbacks are required.

- **Protective Works/Dune Management** - ongoing beach nourishment plus drainage works and revegetation over the eastern 800 m of beach to prevent wave overtopping would cost around $1 000 000 over 50 years (Bay Road $450 000, Sandy Place $600 000), with a present day cost of $700 000. A seawall would be over $2.5 million and could cause a loss of sandy beach amenity.

Based on the above assessment, continuing with and strengthening the existing development controls, including minimum floor levels and building setbacks, would appear to be the best option. Beach nourishment, although not economically justified, may be socially desirable (along Bay Road) and could involve the local community.

10. MALONEYS BEACH

10.1 Coastal Hazard Review

Maloneys Beach is a pocket beach about 1 km long in Chain Bay (within Batemans Bay) between Long Beach and Acheron Ledge (see Figure 1). There are no major identified coastal hazards for the beach although there are some minor problems with *wave overtopping* and *entrance stability*. There is also back beach catchment runoff/coastal inundation flooding of several low lying properties along Maloneys Creek.

Maloneys Beach is the most easterly of the bay beaches, and because of its location faces south and south-west. A stabilised dune has been formed along the beach, with a level of 6.0 mAHD but falling to 3.0 mAHD at the ends. Maloneys Creek has an outlet through the western end of the beach. The creek connects to a large (SEPP 14) freshwater wetland some 800 m upstream (see Figure 8).

The beach is backed by a small urban development with a ground level of approximately 5.0 mAHD, although there are some low lying areas near the wetland at around 3.5 mAHD. There is a beach boat launching area and wooden ramp at the eastern end of the beach which has been designed to accommodate the erosion hazard.

The Vulnerability Study (CRMD,1996) estimates peak 1%, 2% and 5% AEP still water ocean levels at 3.0, 2.9 and 2.8 mAHD respectively. This includes 0.3 m uncertainty, but does not include a climate change allowance. With the addition of a further 2.1 m wave runup estimate, there is the possibility of minor overtopping into the wetland and erosion/inundation of the boat ramp area.

10.2 Coastal Hazard Impacts

Photogrammetric analysis of historical aerial photographs undertaken for the Vulnerability Study identified long term beach recession between 1942 and 1993 at around 0.15 m^3/m/yr, and an envelope for storm bite over the 18 years between 1972 and 1990 of 12 m^3/m. The Vulnerability Study also assessed potential beach recession of 10 m for a 50 year planning period as a result of a 0.2 m Greenhouse sea level rise.

The assessed storm bite is considered to be conservative (i.e. high), because the storm bite allowance includes 18 years of ongoing recession. However, even with this conservative assessment, the Vulnerability Study did not identify a significant property or infrastructure hazard. Storm bite erosion does create a beach scarp which affects the board and chain boat ramp. This can make boat launching difficult until appropriate maintenance reshaping is undertaken.

In the event of overtopping into the wetland channel there is the possibility of salinity damage to the wetland. However, the duration of inundation would be short (during the high tide period only), and would usually be associated with substantial rainfall and runoff. Further, many of the identified wetland plants are at least partially tolerant to brackish water.

Excess catchment runoff associated with high ocean surge levels is known to cause some inundation problems in low lying areas and to some residences along the creek/wetland. An assessment of these problems would require catchment runoff modelling which is beyond the scope of the study.

10.3 Options Assessment

There are no major coastal hazards affecting Maloneys Beach, although flooding of the back beach lagoon area and maintenance problems with the beach boat launching ramp are related to storm waves (see Figure 8).

To assess and evaluate the effects of catchment runoff and elevated ocean levels on the flooding of back beach areas along Maloneys Creek a Flood Study and Floodplain Management Study should be undertaken at a cost of around $40 000. To provide for ongoing use of the beach as a boat launching area, provision would need to be made for ongoing ramp maintenance to reshape the beach and re-lay the ramp after significant storm events. Assuming an average of three maintenance operations per year, the cost of this option would be around $5000 per year.

Because of the minor nature of the coastal hazards identified for Maloneys Beach, no major management options have been recommended. Suggested options could include a flood study of Maloneys Creek and regular maintenance on the boat ramp.

11. HANGING ROCK

11.1 Coastal Hazard Review

For this study the "Hanging Rock" area extends from Batemans Bay boat harbour/marina and Hanging Rock Creek to the dunes behind Corrigans Beach (see Figure 1 and 9). The main coastal hazards identified for this area are *coastal inundation* combined with *local catchment runoff*.

The rock training wall which forms the southern foreshore of the Clyde River entrance channel continues from the boat harbour to Corrigans Beach (see Figure 9). Immediately behind the wall there is an intertidal basin which was originally part of the bay but is now the entrance to Hanging Rock Creek. The basin also forms the boat harbour/marina in the west and a boat launching ramp area in the east near Corrigans Beach. The training wall crest varies between 1.8 and 2.2 mAHD, although there are two navigable openings, one near the marina and the other near the boat ramp.

The area around the basin is very low, with a significant proportion at or below 1.5 mAHD. Along most of the foreshore there is a caravan park/resort backed by a partial levee with a crest height around 2.3 mAHD (see Figure 9). South of the levee there is a residential subdivision of some 100 properties. The Hanging Rock subdivision (including Marlin, Dolphin, Avalon and Tuna Streets) has reportedly experienced flooding during very high tides in the past (PWD, 1989).

Design still water and wave runup levels for the Hanging Rock foreshore can be interpolated from the Oceanic Inundation Study (PWD, 1989) based on theoretical conditions and include an uncertainty factor. A more recent investigation of flooding in the Hanging Rock area (JCA, 1998) examines flooding as a result of local drainage, catchment runoff and ocean inundation assuming a complete levee around the area.

For this Management Plan the estimates for elevated foreshore setup levels and wave climates were reviewed for climate change effects and a range of design events. This information was then used to estimate inundation impacts under current (partial levee) conditions and to review management options for the area.

11.1.1 Foreshore Setup Levels

Elevated foreshore setup levels for the Hanging Rock area were reviewed based on a similar approach to that undertaken for the Beach Road area (refer Chapter 5). Because of the similarities between the two areas in terms of their location along the Clyde River estuary entrance, the same astronomic tide, storm surge, and climate change allowance was applicable as was assessed for Beach Road. However, because of the wider bay and more exposed location the allowances for river flooding, wave setup and wave runup were assessed separately.

Major storm waves with offshore significant wave heights of over 7 m become nearshore waves of over 1.5 m along the Hanging Rock foreshore east of the Boat Harbour and river flooding impacts drop to below 0.05 m (PWD, 1989). A summary of the setup levels is shown in Table 19.

Table 19: Summary of Hanging Rock Foreshore Setup Levels (mAHD)

Location	AEP	Astronomic Tide & Storm Surge	Plus River Flooding	Plus 50 yr Greenhouse	Plus Wave Setup	Cumulative Level
Hanging Rock (Boat Harbour East)	5%	1.4	0.00	0.2	0.2	1.8
	2%	1.45	0.00	0.2	0.25	1.9
	1%	1.5	0.00	0.2	0.3	2.0

Based on the revised assessment, the setup water levels adopted for the Hanging Rock area were 2.0, 1.9 and 1.8 mAHD for the 1%, 2% and 5% AEP events respectively.

11.1.2 Wave Overtopping and Inundation

The adopted setup levels are similar to the existing foreshore wall crest level, and higher than the foreshore caravan park, backshore roadways and general ground level in the Hanging Rock area. Only the partial levee, sections of Beach Road, and the dunes along Corrigans Beach are above the adopted setup level. Therefore, during design conditions waves would overtop the training wall and encroach over both the foreshore and backshore areas.

To determine wave overtopping rates, the procedure identified in the Shore Protection Manual (CERC, 1984) was used (see Table 20). The table also gives the theoretical wave runup should the training wall be raised as part of any future mitigation works.

Table 20: Summary of Wave Overtopping and Theoretical Runup Levels

Location / Wall Length (m)	**AEP**	**Overtopping Flow (m^3/s)**	**Theoretical Runup (m)**	**Estimated Inundation Level**	
				Backshore (mAHD)	**Foreshore (mAHD)**
Hanging Rock	5%	630	1.6	1.8	2.1
	2%	810	1.6	1.9	2.3
700m	1%	1000	1.6	2.0	2.5

Based on the assessed levels, wave overtopping of the training wall would occur during high tide, with intense storm conditions, and would coincide with high catchment rainfall and runoff. Modelling undertaken for the CBD indicated that in the lower catchment (below Beach Road) maximum inundation levels would largely be determined by the very large flows entering the area from the bay, and that these would probably last over an hour (through the high tide period).

However, unlike the CBD, flood levels in the Hanging Rock area would be modified by the presence of the back wall basin, the partial levee, and inflows and outflows through the boat harbour and boat ramp entrances. These factors should produce peak levels in the backshore (Hanging Rock residential) area at around the setup levels identified in Table 19. In the nearshore area (around the basin) overtopping of the wall would produce waves in the basin up to 0.5 m. These waves would be dissipated on the levee. As a result nearshore design water levels including wave effects would be around 2.5 m for a 1% AEP event.

11.1.3 Catchment Flooding

Catchment flooding also has the capacity to affect inundation levels in the Hanging Rock area. A recent flood study (JCA, 1998) examined local drainage as well as the wider problem of Hanging Rock Creek catchment runoff. The study also assessed the impacts of a levee around the Hanging Rock area.

When considering just the local catchment, the study found that the existing drainage was sufficient to carry 1 in 16 year (6% AEP) flows, but that larger events would produce surcharging which would cause shallow ponding over a large part of the residential subdivision. Runoff from the wider catchment was also found to cause flooding of the Hanging Rock subdivision. Levels of over 1.95 mAHD were calculated for Hanging Rock Creek downstream of the Beach Road culverts for a 1% AEP event. The study noted that flooding increased with the downstream ocean level, but did not consider ocean levels beyond 0.9 mAHD.

Runoff from the Hanging Rock Creek catchment also has the capacity to cause local flooding above Beach Road, particularly along Hanging Rock Creek and the golf course. An assessment of flooding above Beach Road was not included in the Hanging Rock study (JCA, 1998), and would require a separate study beyond the scope of this Management Plan.

11.2 Coastal Hazard Impacts

11.2.1 Coastal Inundation/Wave Overtopping

Under existing conditions the Hanging Rock boat launching ramp would be exposed to 0.5 m waves and overtopping flows during design storm events which would damage infrastructure and facilities in this area. Around the foreshore of the basin and in low lying caravan park areas, flooding and wave impacts would inundate properties up to 2.5 mAHD during a 1%AEP event which would require caravans and mobile homes to be evacuated (because they are easily damaged).

Beyond the immediate foreshore area in the Hanging Rock subdivision there are numerous residences with low floor levels which would be inundated up to around 2.0 mAHD by wave overtopping combined with catchment runoff. Because of the generally low ground levels, a large number of residential properties would be inundated.

11.2.2 Catchment Flooding

The actual impacts of local drainage flooding from the immediate catchment were not assessed for the flood study (JCA, 1998), but the extent and depths identified suggest nuisance flooding of roads and yards rather than house flooding.

Flooding from the wider Hanging Rock Creek catchment would be severe during a major event in the area downstream of Beach Road. Potential flood levels are similar to those associated with coastal inundation in the immediate foreshore area, and a combination of both is likely to exacerbate the problem. Evaluation of the combined impacts is beyond the scope of this Coastline Hazard Management Study. Catchment flooding impacts upstream of Beach Road are unlikely to be substantially affected by ocean water levels because of the flow restriction caused by the Beach Road culverts.

11.3 Options Assessment

There are several options and combinations of options available for the Beach Road area including:

- Do Nothing,
- Environmental Planning:
    - voluntary purchase,
- Development Controls:
    - minimum floor levels,
    - building protection (raising floor levels and hazard proofing),
- Protective Works:
    - training walls,
    - wave barriers/levees.

11.3.1 Do Nothing

The main advantage of a “do nothing” approach is that it involves no cost to implement. However, there are potentially very large direct and indirect costs associated with future wave overtopping and inundation damage in the area.

Based on the work undertaken for the residential part of the CBD area, total damages to the caravan park and infrastructure (plus some house flooding) would cause losses in excess of half a million dollars during a 1% AEP event, with an AAD of around $35 000, and with a net present value of $500 000 based on a discount rate of 7% pa over a 50 year planning period.

Note, this figure includes a significant local catchment runoff component, but would be much higher for a major catchment runoff event (with or without coastal inundation impacts).

11.3.2 Voluntary Purchase

While voluntary purchase is theoretically suitable as a mitigation measure for this area, the number of possible residential and tourist accommodation properties affected by wave overtopping/coastal inundation is large and the number affected by catchment runoff is even larger. Because of this a program of voluntary purchase would need to include a large number of residential properties as well as the caravan park. The cost of such a program would probably exceed $5 000 000 and would not be economically or socially feasible.

11.3.3 Minimum Floor Levels

Council has a policy on flood liable developments (Appendix C) which sets a minimum residential floor level of 3.0 mAHD for the Hanging Rock area. This level is based on the Ocean Inundation Study (PWD, 1989) and includes a 0.5 m freeboard. The policy has been applied to new development since 1988. It also provides 0.5 m freeboard over the 1% AEP level determined for this study.

Council also has a policy for caravan parks implementable at change of ownership or use (Appendix C). This policy has different risk considerations for permanent and mobile vans and cabins. For Hanging Rock the policy requires minimum floor levels for:

- Permanent/long term cabins - > 3.0 mAHD (1% AEP plus 0.5 m),
- Park owned vans & annexes - > 0.6 & 0.1 m above ground level,
- Private cabins/mobile homes - > 3.0 mAHD (1% AEP plus 0.5 m),
- Private towable vans & annexes - > 0.6 & 0.1 m above ground level,
- Tents - No minimum level.

In addition the policy requires no permanent/long term sites in recession zones, and that park vans/cabins in the hazard zone must be relocatable.

The implementation of these policies has reduced the potential level of damages during a major overtopping event. However, there remain a substantial number of properties built before the policies started with floor levels below the set standards. Further, there are park owned dwellings which are designed as moveable cabins, but which would probably be very difficult to raise during a major storm event.

Based on the 1% AEP ocean inundation water levels determined for this study, the existing interim flooding policy would provide a freeboard allowance of around 1.0 m for residential development in the Hanging Rock subdivision. (Note, consideration of catchment flooding effects from Hanging Rock Creek would significantly reduce the freeboard). The 1% AEP inundation level assessed for cabins in the low lying foreshore area would provide a 0.5 m freeboard, the same as Council's policy.

11.3.4 Building Protection

This option involves identifying at-risk development and reducing potential damages by implementing site specific measures such raising floor levels or inundation proofing. This option has the advantage of providing protection where it is needed. However, there are limitations to the types of residences that can be raised (generally only non-brick, single storey houses on piers). There are also limits to the height caravan park cabins can be raised from a practical and aesthetic view. Caravans are usually raised by filling the site.

To meet Council's existing floor level policy, around 1.5 m or 40 000 m^3 of fill material would need to be added over most of the caravan park site. Practically this would be very difficult, but financially may be theoretically feasible if a local borrow source was available (such as the boat harbour). On this basis the cost of site raising could be around \$500 000.

The estimated cost to raise an average sized house is \$40 000. This does not include potential damage to infrastructure and property not raised (such as garages, pools, laundries and gardens), and caravans. Based on raising a minimum of say 15 houses, the cost of implementing this option would be at least \$600 000. Including site filling over part of the caravan park would make a total of around \$1 100 000 for this option.

11.3.5 Training Walls

Overtopping and inundation damage to foreshore development and low lying backshore areas may be restricted by constructing a physical barrier. The existing rock wall east of the boat harbour to Corrigans Beach could be raised to a sufficient height (3.2 mAHD, at least) with armour stone of around 1.0 to 1.5 tonnes (see Figure 9a). This would prevent most wave overtopping, but would not stop waves entering the basin through the navigable openings.

Provided the height of the existing wall can be raised without widening the wall base, the estimated cost for say 600 m of the total 900 m from the boat harbour to Corrigans Beach is approximately \$500 000, requiring some 4000 tonnes of rock. However, if widening of the wall is required, the cost would be around \$1 500 000.

11.3.6 Wave Barriers/Levees

The recent flood study (JCA, 1998) examined the effects of raising the caravan park area around the basin foreshore to 2.8 mAHD, and extending this to Beach Road along Hanging Rock Creek and to the high ground behind Corrigans Beach, to form a levee around the Hanging Rock subdivision. The study determined that the concept was feasible but would also require significant upgrading of the local drainage.

Raising the foreshore area around the basin would require around 5 000 m^3 of fill material and upgrading of the local drainage would require larger drainage channels and new floodgates. The estimated cost of this work would be around $300 000.

To complete the levee around the Hanging Rock subdivision would involve construction of a wall approximately 500 m long requiring some 6 000 m^3 of fill material as well as modifications to existing infrastructure and accessways. The estimated cost of this aspect of the project is approximately $450 000, bringing the total project to around $750 000.

11.4 Options Summary

The main coastal hazards in the Hanging Rock Creek area are coastal inundation and wave impacts in the immediate foreshore area and coastal inundation combined with catchment runoff in low lying areas along Hanging Rock Creek. The assessment of management options found (see Summary Table 8 and Figures 9 and 9a):

- **Do Nothing** - the present value of potential coastal hazard damages would exceed $500 000 over a 50 year planning period, but damages would be much higher if major catchment runoff flooding was also included.
- **Environmental Planning** - the extent of existing development and the large number of developments affected means that options such as voluntary purchase would cost over $5 million and so are unlikely to be economically or socially feasible.
- **Development Controls** - existing minimum floor level freeboards are probably adequate in the immediate foreshore zone (0.5 m) although wave impact controls are required. In backshore areas levels are largely determined by catchment runoff. The cost of implementing development controls, such as house raising or site filling, would be over $1.0 million for a minimum program.
- **Protective Works** - raising and upgrading the existing training wall would prevent most wave overtopping and impact damage but could not prevent inundation because of the navigation openings. The cost would vary depending on the construction method but would be less than $1.5 million. Constructing a levee to protect the caravan park alone would cost around $300 000, while a larger levee around most of the developments threatened by coastal hazards would cost around $750 000.

Based on the above, continuing with the existing building protection development controls and the construction of a levee around the caravan park area only would appear to be the most viable option. Construction of a levee around all of the Hanging Rock subdivision may also be cost effective option if catchment flooding were included in the damages assessment.

12. CORRIGANS BEACH

12.1 Coastal Hazard Review

Corrigans Beach is one of the southern zone beaches (see Figure 1). It lies between the Batemans Bay entrance training wall in the north and Observation Head in the south (with a zeta shaped curve at the southern end). Construction of the Clyde River entrance training wall in the late 1890's and a seawall extension in 1988, has resulted in substantial accretion along the beach. The accreted (prograded) areas are potentially subject to storm *beach erosion* and significant *wave overtopping* and *coastal inundation* problems.

Over most of its 2 km length the beach dunes have a crest level of over 3.0 mAHD, although this drops to below 2.5 mAHD toward the southern end. The areas at most risk from coastal hazards are the three caravan parks located along the southern and central sections of the beach (see Figure 10). These parks have ground levels typically around 1.6 mAHD, and have a significant number of permanently occupied beach front cabins. The larger of the two southerly parks has cabins within 15 m of a rock revetment wall which forms the high tide mark. The wall has a crest level of around 2.5 mAHD and provides some additional erosion protection.

The coastal processes of Corrigans Beach were examined in a Coastline Processes Study undertaken by Public Works (PWD, 1989a), and have also been reviewed or commented upon in the Vulnerability Study, the Surfside Beach Erosion Study (PBP, 1997) and the Surfside Resort EIS (SKP, 1986). Catchment flooding has been examined in the Joes Creek Flood Study (WP, 1989a). For this Management Plan a review of the previous studies was undertaken with particular reference to coastal/entrance processes and the likely coastal inundation levels for storms up to the 1% AEP event including climate change effects.

12.1.1 Beach Erosion

The Vulnerability Study identifies a storm bite potential of 50 m^3/m length along the southern $^2/_3$ of the beach, except in the section with rock protection. This assessment was based on comparison of the 1972 and 1990 aerial photographs. The Corrigans Beach Coastline Processes Study (PWD, 1989a) identified a smaller storm bite potential for the southern section of the beach, and suggested that the observed erosion was the result of sand movement/beach realignment associated with the 1988 training wall extension (not storm bite).

If the observed erosion was part of the beach realignment process (as would seem reasonable), then the assessed erosion level would no longer be applicable, because (based on shoal morphology) sand buildup and movement on to the beach appears to have slowed substantially over recent years. On this basis a significantly reduced level of storm bite may well be applicable, particularly for the southern section of the beach, but assessment of an appropriate level would require a more detailed coastal processes study.

For the purposes of this Management Plan a slightly reduced amount of 40 m^3/m length has been adopted for storm bite erosion.

12.1.2 Shoreline Recession/Accretion

As already mentioned, Corrigans Beach has accreted (prograded) as a result of training wall construction. The average annual increase in beach width, over the 51 years from 1942 to 1993, has varied from around 2 m^3/m length in the southern portion to over 9 m^3/m near the training wall.

Based on existing conditions and assuming that beach accretion and realignment has now slowed, and that there will be no further training wall extensions, it is reasonable to now adopt a zero accretion rate for the beach. However, this assessment does not include any recession associated with climate change sea level rise.

The Vulnerability Study examined the potential impacts of climate change sea level rise for the "most likely" scenario, and determined a total recession of 6 m over a 50 year planning period. This value has therefore been adopted as the 50 year shoreline recession value for the Management Plan.

12.1.3 Coastal Inundation

The Vulnerability Study found that the peak 1%, 2% and 5% still water setup levels along Corrigans Beach are 2.6, 2.5 and 2.4 mAHD respectively. Discounting for the 0.3 m uncertainty factor, but including a 0.2 m general sea level rise for climate change effects gives levels of 2.5, 2.4, and 2.3 mAHD respectively. For a design inshore wave height of 1.5 m the Vulnerability Study estimated wave runup on the beach to be around 0.9 m, with a possible 4.0 m at the rock revetment.

Based on the assessed levels, wave overtopping of the beach dunes could be expected, particularly towards the southern end in the vicinity of the rock revetment. Using the procedure identified in the Shore Protection Manual (CERC, 1984), during a 1% AEP event waves would overtop the dune and revetment wall at rates between 0.4 and 1.1 m^3/s per metre foreshore length depending on the varying conditions. This would be equivalent to a peak overtopping rate of around 400 m^3/s into the back beach area.

12.2 Coastal Hazard Impacts

12.2.1 Beach Erosion

There is little information on the structural stability of the seawall near the larger of the two southern caravan parks. However, based on observations, the wall appears unlikely to withstand a major storm event. The probability of failure is also likely to rise with time, as the sand at the toe of the wall and along the adjoining beach recedes due to increasing climate change impacts. Therefore, as a result of the combined effects of climate change sourced recession and storm bite it is possible that the first row of cabins along this caravan park will be threatened by beach erosion within a 50 year planning period.

Cabin developments at the central caravan park are set back from the beach foredunes by over 40 m. As a result the likely erosion hazard impact at this site within a 50 year planning period is very small.

12.2.2 Coastal Inundation

During a 1% AEP storm event the level of inundation in the immediate foreshore area would be around 2.5 mAHD including some wave effects, but not including wave impacts or localised wave runup. Away from the foreshore area overtopping flows would drain towards Joes Creek.

Wave overtopping and inundation at the estimated levels would cause extensive back beach flooding and wave impact damage to development in the immediate foreshore area. Because of the low lying nature of the existing caravan parks and their proximity to the foreshore, these facilities would be particularly susceptible to damage during design storm events.

During more extreme events flooding of the back beach area would be extensive. To assess this problem a flood study of Joes Creek has been undertaken (WP, 1989a). This study considered the effects of major catchment inflows together with elevated ocean levels at 2.25 and 2.55 mAHD. The study found that even moderate catchment runoff (5% AEP) during major elevated ocean events caused flood levels of around 2.8 mAHD downstream of Beach Road.

12.3 Options Assessment

The Vulnerability Study identified two management options, the setting of appropriate floor levels, and retreating facilities if threatened by erosion/recession. However, there are several options and combinations of options available for the Corrigans Beach area including:

- Do Nothing,
- Environmental Planning:
  - Voluntary purchase,
- Development Controls:

    - minimum floor levels,
    - building protection (raising floor levels and hazard proofing),
    - planned retreat,
- Dune Management,
- Protective Works:
    - seawalls,
    - beach nourishment.

The latter two options apply particularly to the southern third of the beach, to prevent wave overtopping and safeguard against storm bite erosion.

12.3.1 Do Nothing

The major consequences of adopting the "do nothing" option would be coastal inundation and the possibility of foreshore erosion, particularly of the southern caravan park. Based on the findings of the Vulnerability Study wave overtopping into the southern caravan park area could be expected on average once every 10 years or less with climate change effects, and into the other parks during major storm events. Assuming some modifications to park management in response to the hazards, damages of around $450 000 could occur during a major event, with an AAD due to inundation of around $18 000 with a net present value of $250 000 based on a discount rate of 7% over a 50 year planning period.

12.3.2 Voluntary Purchase

The developments most affected by a major storm/ocean inundation event would be the caravans and mobile homes sited along the immediate foreshore of much of the southern area. However, these developments are crucial to the economic viability of the caravan park. As a result, any voluntary purchase program for this area would probably involve closure/amalgamation of one of the caravan parks.

The total cost of such actions would include, not only the park closure but also the social and economic impacts associated with the loss of a significant local tourist development. The resulting costs are difficult to assess, but are likely to be many times greater than the damages estimated for the "do nothing" option. However, voluntary purchase may become a more desirable and feasible option in the future as hazards increase with climate change effects and problems become more immediate.

12.3.3 Minimum Floor Levels

Council has a policy for new development and major redevelopment in flood liable areas (Appendix C) which sets a minimum residential floor level of 3.3 mAHD for the Corrigans Beach area. This level is based on the Ocean Inundation Study (PWD, 1989) and the Joes Creek Flood Study (WP, 1989a), and includes a 0.5 m freeboard. The policy has been applied to new development since 1988.

Council's existing policy for caravan parks implementable at changes of ownership or use (Appendix C) includes different risk considerations for commercial and private property. For Corrigans Beach this policy requires minimum floor levels for:

- Permanent/long term cabins - > 3.3 mAHD (1% AEP plus 0.5 m),
- Park owned vans & annexes - > 0.6 & 0.1 m above ground level,
- Private cabins/mobile homes - > 3.3 mAHD (1% AEP plus 0.5 m),
- Private towable vans & annexes - > 0.6 & 0.1 m above ground level,
- Tents - No minimum level.

In addition the policy requires no permanent/long term sites in recession zones, and that park vans/cabins in the hazard zone must be relocatable.

Based on ocean inundation considerations alone, the existing policy levels provide a freeboard allowance of around 0.8 m over the 1% AEP foreshore levels assessed for this study. However, after inclusion of catchment runoff the freeboard would be the same as the policy levels. However, there are semi-permanent developments in the immediate foreshore zone, and a number of cabins which are theoretically movable but which would probably be very difficult to raise during a major storm event. These developments would probably suffer extensive damage during a major storm event.

The costs of undertaking this option are those associated with ongoing implementation, and the damages associated with the continuing inundation hazard.

12.3.4 Building Protection

This option involves identifying at-risk development and reducing potential damages by implementing site specific measures such raising floor levels or inundation proofing. This option has the advantage of providing protection where it is needed. However, there are limitations to the types of residences that can be raised (generally only non-brick, single storey houses on piers). There are also limits to the height caravan park cabins can be raised from a practical and aesthetic view. Caravans are usually raised by filling the site.

To meet Council's existing floor level policy, over 1.5 m or 80 000 m^3 of fill material would need to be added to the caravan park sites. Practically and financially this would be very difficult and even with a local borrow source (such as the northern accreted end of the beach) the cost would exceed $1 000 000.

The estimated cost to raise an average sized house is $40 000. This does not include potential damage to infrastructure and property not raised (such as garages, pools, laundries and gardens), and caravans. Based on raising a minimum of say 5 houses, the cost of implementing this option would be at least $200 000. Including site filling over parts of the caravan parks would make a total for this option of over $1 200 000.

12.3.5 Planned Retreat

An option identified in the Vulnerability Study, which could be considered as circumstances require, is to move threatened structures away from areas of beach erosion and recession. Note however, this option does not address the other major problem of ocean inundation.

Relocating structures within the existing boundary of the southern caravan park would be difficult because of the intensity of the existing development. One possible method would be to move more permanent sites back from the first row as part of ongoing redevelopment requirements. This would allow the foreshore to be utilised as open space and overflow camping as required.

12.3.6 Seawall

Extending and raising the existing seawall along the southern beach foreshore could provide ongoing protection against beach erosion, and overtopping and inundation. This option involves construction of an extended revetment wall, approximately along the alignment of the existing wall (see Figure 10a).

The wall would consist of a geotextile filter cloth on top of which two layers of 1.7 tonne armour stone would be placed. Suitable stone from the existing wall could be reused. The finished wall would have a crest level of over 4.5 mAHD to prevent overtopping. In order to prevent scour damage, the new wall would have a toe depth at least –1 mAHD.

The cost of constructing a 1000 m revetment wall requiring 40 000 tonnes of rock, consisting of two layers of armour stone, with a crest of 4.5 mAHD and to a depth of –1 mAHD is of the order of $2 500 000. A partial wall protecting only the southern caravan parks would be more than $1 000 000 and would not fully address the problem of ocean inundation. Further, additional costs associated with the loss of a sandy beach amenity in front of the wall and ocean views behind the wall are major but have not been included.

12.3.7 Dune Management/Beach Nourishment

Raising the crest level of the dune over the southern 1200 m of the beach could substantially eliminate coastal inundation (see Figure 10a). The approximate volume of sand required to raise the dune to around 3.5 mAHD to prevent major overtopping is 15 000 m^3, not including maintenance after major storms. If the sand could be sourced from the accretion zone near the training wall at the northern end of the beach, which would appear likely, the cost to carry out this option would be around is $200 000. Removing sand from this area has the added benefit of helping prevent bypassing of the entrance training wall and so reduces long term adverse impacts on the nearby regional boat ramp. If the sand were to be imported from a commercial source the cost would exceed $500 000.

If sufficient sand were to be placed on the beach to minimise the risk from beach erosion some 25 000 m^3 of suitable sand would be required initially plus ongoing maintenance after storms. The estimated cost of this option would initially be around $350 000 if the sand were sourced locally. The net present value (based on a 7% discount factor) of initial works and ongoing nourishment of 10 000 m^3 every 10 years would be around $650 000. Note, some camping sites in the southern park may be covered with sand as a consequence of undertaking this nourishment option.

12.4 Options Summary

As discussed above, the coastal hazards which impact on Corrigans Beach are beach erosion with some climate change induced recession, and coastal inundation of low lying back beach (caravan park) areas. There is also potential for some coastal inundation of residential areas when combined with runoff from the Joes Creek catchment. The assessment of management options found (see Summary Table 9 and Figures 10 and 10a):

- **Do Nothing** - the present value of the potential coastal hazard damages would be around $250 000 over a 50 year planning period (assuming some changes to park management and not including catchment induced flooding).

- **Environmental Planning** - voluntary purchase is unlikely to be feasible in the short term because of the need to purchase and close down one of the caravan parks (cost >$500 000) and the resultant social and economic impacts. However, this option may become feasible and desirable as climate change effect increase.

- **Development Controls** - existing minimum floor levels are determined more by catchment runoff than coastal inundation and appear reasonable. The cost of a program of building protection, including caravan site filling and minimal house raising, would exceed $1.2 million. Planned retreat would help address the erosion problem but not the problem of coastal inundation. Planned retreat could be combined with minimum floor level requirements to provide a full option.

- **Protective Works** - construction of a seawall or beach nourishment with local sand to protect development from inundation and erosion would cost $2.5 million and $650 000 respectively. The cost of a partial seawall to protect against erosion only would be over $1.0 million. Beach nourishment to prevent overtopping only would be around $700 000 for local sand. The locals and beach nourishment options also assist with maintenance of the nearby regional boat ramp.

Based on the above, a continuation of the minimum floor level policy with a planned retreat policy, and no major landfill building protection would appear to be the most feasible options, although voluntary purchase may be feasible and desirable in the future as climate change impacts increase.

13. CASEYS BEACH

13.1 Coastal Hazard Review

Caseys Beach is the most southerly of the Batemans Bay beaches examined for this study (see Figure 1). The beach has a revetment seawall running most of its length. The main coastal hazards are *wave overtopping* of the wall and associated wall stability questions including possible failure during major events.

Caseys Beach is an 800 m long pocket beach between Observation Head and Sunshine Bay. It faces the north east and is exposed to a low to medium wave climate. After major storm erosion in 1974 and again in 1975, a revetment wall was constructed along what was the beach foredune (see Figure 11). The purpose of the wall is to protect Beach Road and the residential properties which face the beach, and a Sewage Pumping Station at the southern end. Short Beach Creek enters the ocean in the southern corner of the beach.

Details of the seawall revetment structure are not available, but anecdotal evidence and newspaper photographs and reports suggest that the wall is a mix of sand and quarry rubble of varying sizes. The seaward face of the wall was topped with 600 to 900 mm granite rocks after a storm in the early 1990's. Wave reflections from the wall have accelerated sand loss from the beach and reduced the beach amenity.

The wave overtopping hazard has been addressed on the basis of elevated ocean water levels and storm wave climate data prepared for the Oceanic Inundation Study (PWD, 1989) and the Vulnerability Study. A Flood Study of Short Beach Creek has also been undertaken (WP, 1989). The stability of the revetment wall has been addressed based on the available anecdotal information. Questions regarding coastal erosion and shoreline recession are no longer relevant for Caseys Beach because of the wall.

13.1.1 Coastal Inundation

Construction of the revetment wall and progressive upgrading of Beach Road has resulted in Caseys Beach having no dune system, and there being no buffer between the rock wall and the road. The height of the road along the beach is between 3.0 and 4.0 mAHD. The road is around 3.0 mAHD at the bridge over Short Beach Creek, but rises to over 4.0 mAHD along the northern part of the beach.

The Vulnerability Study found that the peak 1%, 2% and 5% AEP still water levels along Caseys Beach are 2.7, 2.7 and 2.6 mAHD respectively including a 0.3 m uncertainty factor. Discounting for the uncertainty factor, gives a short term 1% storm level of 2.4 mAHD and a longer term level including a 0.2 m climate change allowance of 2.6 mAHD.

The Vulnerability Study also determined a nearshore design storm wave height of 2.5 m for the central portion of the beach, with a theoretical wave runup estimate of 4.6 m based on the wall being impermeable. Clearly, a total theoretical runup level in excess of 7 mAHD implies extensive inundation damage could occur to Beach Road and the properties fronting the road, even if the assumed conditions do not fully apply.

The Short Beach Creek Flood Study found the 1% AEP peak flow in the creek to be around 105 m^3/s, and flood levels along the creek in the back beach area to be around 2.5 mAHD. The study used a steady 2.43 mAHD ocean level as the downstream storm tide condition, but did not examine the impacts of wave overtopping on flood levels.

Estimation of overtopping rates using the procedure identified in the Shore Protection Manual (CERC, 1984) is difficult because of potential wave energy dissipation in the nearshore surf zone and on the revetment wall. However, based on reasonable assumptions with waves up to 0.3 m in height crossing the road, a total of around 150 m^3/s could be flowing across Beach Road during a 50 year storm event. Much of this flow would enter Short Beach Creek west of the properties on Beach Road via local drainage pathways.

Overtopping flows of the order of those estimated for a 50 year storm event are of a similar magnitude to those estimated for 1% AEP peak catchment runoff flows in the Flood Study. On this basis, wave overtopping would have the potential to substantially affect flood levels in the back beach area and along Short Beach Creek. To accurately determine these levels would require a catchment flood study which is beyond the scope of this study.

13.1.2 Seawall Stability

Because of the lack of reliable information on the physical structure of the existing rock revetment wall it is not possible to determine the level of protection provided by the revetment wall. However, based on the wave climate conditions identified in the Vulnerability Study, the wall, if designed for minimal damage during a 2% AEP storm, should have:

- two layers of armour stone weighing approximately 1.7 tonnes, or around 1.0 m in size,
- a crest height of around 5 mAHD, to prevent overtopping,
- a toe depth of around -1 mAHD, possibly with a rock mattress apron,
- well formed filter rock layers or a heavy duty geotextile filter cloth layer to prevent leaching of the sand base.

The available information confirms that the existing revetment wall does not meet the above specifications. The wall does not have two layers of armour, and the armour that it does have is around 1.0 tonne, which would suffer around 50% damage during a design storm event (CERC, 1984). Further, the wall could be undercut by wave scour because of inadequate toe depth, or collapse due to the leaching of the fines from behind the wall because of lack of an adequate filter medium.

13.2 Coastal Hazard Impacts

13.2.1 Coastal Inundation

Wave overtopping of the revetment wall would close Beach Road and inundate some low lying properties without wave barrier fencing fronting the road near the bridge up to the John Street intersection. Council's existing minimum floor level for residential properties is 3.3 mAHD. Implementation of this policy over recent years has reduced the likely extent of damages. However, flood damage and probably wave impact damage to at least 14 properties and 7 residences could still be expected. Extensive damage would occur to picnic facilities and infrastructure at the northern end of the beach, whilst the sewage pumping station at the southern end near the Short Beach Creek bridge would be submerged.

Flood levels along Short Beach Creek would be raised by the inflow of wave overtopping flows. As already mentioned assessing the effects of these inflows on design flood levels along the creek would require a catchment flood study beyond the scope of this Coastline Hazard Management Plan.

13.2.2 Seawall Stability

Failure of the existing revetment wall would inevitably damage Beach Road because of the proximity of the wall to the road. The extent of the damage would depend on the intensity and duration of the storm and Council's capacity to respond. An intense storm at night on a weekend would probably cause major seawall loss and close the road. Damage from a lesser storm during a week day may be minimised by emergency rock dumping. Either way, the amenity of the beach would be substantially lost.

Note, although Beach Road is a major infrastructure facility, it is no longer the major coastal access way south of Batemans Bay. For this reason it is less critical as a transport corridor than prior to the construction of George Bass Drive.

13.3 Options Assessment

The Vulnerability Study identified three management options, do nothing, nourish the beach and raise and strengthen the revetment (and protect the pumping station). However, there are several options and combinations of options available for Caseys Beach including:

- Do Nothing,
- Development Controls:
  - building protection (hazard proofing),
- Protective Works:
  - seawalls,
  - beach nourishment,
  - offshore breakwaters.

13.3.1 Do Nothing

As discussed, the existing revetment wall appears inadequate. Adopting the “do nothing” option would lead to further deterioration of the wall and future damage to Beach Road. Failure of the revetment wall (via overtopping, undercutting by wave scour or collapse due to leaching of fines) is a real possibility under a “do nothing” scenario. As a consequence, Beach Road is also likely to be damaged.

Storm bite erosion will continue to impact on the beach amenity which is already significantly depleted. During overtopping events the road will need to be closed and properties facing the road will suffer wave impact and inundation damage.

The total cost and recurrence interval of damage for a major storm event is difficult to assess without a detailed historical record and accurate survey and construction information. However, assuming an overtopping/damage event occurrence every 10 years based on anecdotal information, infrastructure and residential storm damages totalling around $250 000 could occur with present day damages for a 50 year period of around $450 000 (assuming a 7% discount factor).

13.3.2 Building Protection

To help minimise the damage to residences and infrastructure, and the inconvenience caused by wave overtopping, it would be possible to construct a wave deflection barrier along the western side of Beach Road which would divert flows along the road and into Short Creek (see Figure 11a). This structure would be about 500 mm above the road pavement height and could be incorporated into the road side gutter/footpath or property boundary fences so as not to prevent pedestrian access. The estimated cost of the barrier and associated works would be around $100 000.

Overtopping and damage to Beach Road would not be prevented with this option. Therefore, appropriate warning signs and emergency repair procedures would need to be formulated and put into place in anticipation of future events. These emergency procedures could include the progressive raising and strengthening of the revetment wall. The cost of this option is difficult to estimate, but could result in expenditure of around $150 000 after say every 10 year storm event (totalling >$600 000 over 50 years). This would have a net present day cost of around $300 000 using a 7% discount rate.

13.3.3 Beach Nourishment

This would involve reinstating a dune system and sandy beach amenity in front of the revetment wall and extending the beach area via the importation of sand, probably using a large dredge. Possible sources of sand are:

- the bed of the bay,
- the marginal shoal along the Clyde River entrance,

- the northern end of Corrigans Beach where accretion has occurred behind the training wall.

The estimated cost to reinstate dunes to a level of 4.5 mAHD and extend the beach 150 m in to the surf zone is $2 000 000, assuming a local sand supply in the order of 250 000 m^3 spread along the 800 m of beach (see Figure 11a). The stability of a dune system located in front of revetment wall and the sand source will require considerable detailed investigation should this option be pursued.

13.3.4 Seawalls

This option essentially involves the construction of a new revetment wall to an acceptable standard, along the alignment of the existing wall. The wall would consist of a reformed core covered by a layer of geotextile filter cloth on top of which two layers of 1.7 tonne armour stone would be placed. Suitable stone from the existing wall could be reused. The finished wall would have a crest level of 5 mAHD, which corresponds to a height of between 1 and 2 m above Beach Road. In order to prevent scour damage, the new wall would have a toe depth of at least –1 mAHD.

To protect the sewage pumping station and allow for the Short Beach outlet the wall would need to extend up both sides of the creek to the bridge. To provide a setback from the edge of the road, the new wall would encroach further on to the beach.

The estimated cost of constructing a 800 m revetment wall requiring 30 000 tonnes of rock, consisting of two layers of armour stone, with a crest of 5 mAHD and to a depth of –1 mAHD, with appropriate drainage and leaching controls, is of the order of $3 000 000.

13.3.5 Offshore Breakwaters

As an alternative to raising and strengthening the revetment wall an offshore breakwater/reef could be constructed at around the -5.0 mAHD contour level, or around 200 m from the existing beach face.

To reduce the wave climate on Caseys Beach the reef crest level would need to be at around -1.0 mAHD minimum and extend shore parallel for most of the beach. Large armour rock (or geotextile bags filled with sand) would be required. The wave climate at the reef would be greater than that predicted for the beach. Further, bed protection would be required on both sides of the reef to prevent scour due to wave stirring.

A preliminary estimate of this option indicates that some 50 000 tonnes of rock (or sand bags) in excess of 2.0 tonnes would be required for the reef, with an estimated cost of over $5 000 000.

13.4 Options Summary

The main hazards affecting Caseys Beach are wave overtopping of the existing seawall and the structural stability of the wall revetment under wave attack. The assessment of management options found (see also Summary Table 10 and Figures 11 and 11a):

- **Do Nothing** - the present value of the potential coastal damages are around $450 000 over a 50 year planning period plus significant amenity and social losses.

- **Development Controls** - the cost of hazard proofing properties from inundation would be around $100 000 using a wave deflection barrier/roadside gutter, but this would not prevent damage to the existing seawall and road. Ongoing seawall and road repair, and upgrading is estimated to cost $150 000 per 10 year event (total >$600 000 over 50 years) and have a present value of around $300 000.

- **Protective Works** - reforming of a dune/sandy beach along the seawall would cost around $2.0 million and requires more investigation to assess its engineering feasibility. Reconstructing the seawall to a standard sufficient to prevent damage would cost around $3.0 million. Constructing an offshore breakwater/reef would exceed $5.0 million.

Based on the above, ongoing maintenance/upgrading of the seawall and Beach Road is economically the most feasible solution, and when combined with the construction of a wave deflection barrier, probably provides an adequate social solution.

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APPENDIX A: RUBICON MODEL

A1. INTRODUCTION

HD-system RUBICON was developed by Haskoning BV and Delft Engineering Software. It can be used for studying a wide range of hydraulic engineering problems, such as:

- flood wave propagation through channels, rivers, floodplains and reservoirs,
- tidal flow in rivers and estuaries,
- effects of structures in channel systems,
- sediment movement in rivers and estuaries,
- optimum design and operation of irrigation and drainage systems,
- entrance breakout through an erodible beach berm,
- wave propagation in hydropower systems,
- wave propagation resulting from dam failures,
- hydraulic parameters in water quality studies.

Modelling is based on the full de Saint-Venant equations solved with a highly accurate and efficient modification of Preissmann's implicit finite difference scheme. It is very flexible in specifying external and internal boundary conditions. The user can select from a number of system elements to simulate complex flow over floodplains or define structures at any point of the channel system, such as weirs, gates, culverts, siphons, spillways, sluices, storm surge barriers, dykes, etc.

Limitations are the accurate simulation of super-critical flow and two-dimensional flow situations where the convective momentum terms play a significant role.

Important objectives during the design of the program were to make it a user-friendly system, which would minimise the time required for data preparation, and formulate the system in a modular way to facilitate addition of enhancements.

This is exemplified by the following features:

- programs written in Fortran with the source code made available,
- separate processing for input, execution and output sub-systems,
- extended free format data input, including comments,
- possible to add user defined sub-routines and functions,
- all user-defined model elements (channels, structures, etc.) addressed by names,
- automatic generation of computational grid and element numbers following user's directives,
- use of special information symbols to minimise input effort,
- extensive checking of input data,
- continuation of input processing after detection of errors,
- restart facilities in model execution,
- possible generation of output at any point of the channel system.

The original suite of programs has been extensively modified by Webb, McKeown & Associates. A layout of the current RUBICON modelling system is given as Figure A1.

A2. SYSTEM ELEMENTS

The following range of model elements are available:

- branches,
- cross-sections,
- nodes,
- structures,
- gridpoints,
- lateral flows.

Branches are used as schematised elements for:

- rivers,
- channels,
- estuaries,
- ocean entrances,
- connections between floodplain cells,
- closed conduits.

At the branch limits, *Nodes* are included to provide for:

- free branch ends,
- branch connections,
- floodplain cells.

A single node can connect any number of branches. A boundary condition can be applied at a free branch as a function of:

- height,
- flow,
- critical outflow,
- or any user defined parameter.

Gridpoints are located along branches and have an associated *Cross-Section* which defines the topography. *Structures* can be defined at any place along a branch and basically they provide a relationship between the discharge and upstream/downstream water levels. The definitions of structures are extremely flexible and includes culverts, free overflow structures, submerged structures, underflow structures or local loss structures. It is also possible to simulate complex gate opening/closing structures or pumps. The model also provides for a user-defined structure to be included as a sub-routine. This allows complete freedom in defining the structure.

Culverts are modelled using the approach adopted by Boyd (Reference A1). Culverts are checked for outlet and inlet control and the lesser flow is adopted. Box or pipe culverts can be modelled as well as the shape of the wing-wall, Manning's 'n', slope and other culvert characteristics. Weirs are input as a weir type formulae and are generally represented as a series of horizontal steps with appropriate 'C' values. Ocean entrance berms are represented as trapezoidal sections which erode using the weir formula and appropriate sediment movement equations.

Inflows are generally input at the upstream nodes as a flow versus time function. However *Lateral inflows* can also be included as a flow versus time function at any location along a branch.

A3. OUTPUT

Output from RUBICON is extremely comprehensive including:

- maximum profiles - height, flow or velocity. The time of the peak is provided as well as the value of the other parameters,
- output at every time step of height, flow and velocity which can be represented as a dynamic profile,
- time functions of height, flow, velocity, area, width and a large number of other hydraulic parameters,
- the above data can be used to provide rating curves, hazard diagrams and a number of other plots.

The output can be provided on a screen, disk, hard copy or plot.

The output is output in a format compatible with the HGRAPH (or other) graphics package. This permits extremely high quality output to be provided on a range of devices.

A4. REFERENCES

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APPENDIX B: DESCRIPTION AND ASSESSMENT OF INUNDATION DAMAGES

B1. DESCRIPTION AND ASSESSMENT OF INUNDATION DAMAGES

B1.1 Methodology

Inundation damages for Batemans Bay CBD have been based on the results of three flood studies (References B1, B2 and B3). The methods developed from these flood studies for assessment of flood damages are also adequate for assessment of ocean inundation damages.

B1.2 General

Results from the three Flood Studies and the floor level survey have been used to identify the number of buildings inundated above floor level for various design events. A list of habitable (or work floor level for non-residential buildings) floor levels were obtained and each residential property the underfloor level was assumed to be 0.5 m lower than the habitable floor level. The underfloor level reflects damage to the grounds, garage, etc. The range of flood damages are categorised in Table B1.

There are no records for the study area documenting damage to public property from previous floods. The following is a summary of the expected damages.

Rock Wall Revetment

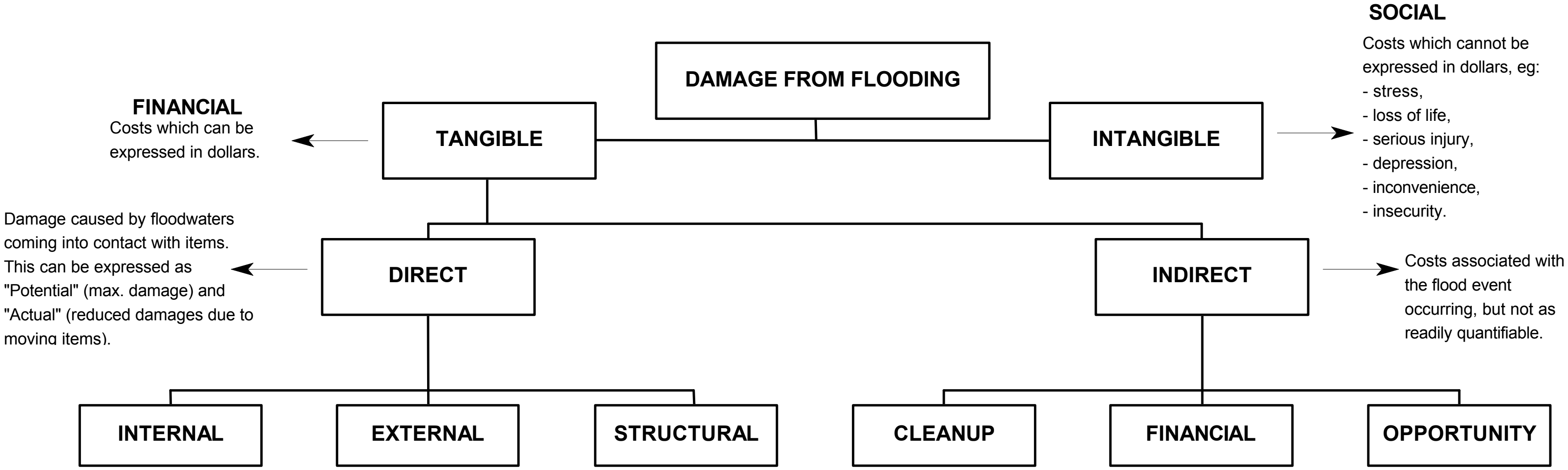
Ocean storm events capable of overtopping the rock wall revetment are likely to result in significant structural damage. Structural damage to the revetment is primarily a result of scouring rather than wave impact.

Roads

Structural damage to road surfaces is likely if the rock wall revetment is overtopped. Roads backed by the revetment are subject to significant structural damage due to loss of road surface and possible road collapse. Overtopping will also contribute to minor accidents, with the possibility of cars being swept off the road surface.

A further cost to the road system results from intangible issues such as inconvenience, worry, delays, risk to life, etc.

Table B1: Flood Damages Categories



	INTERNAL	EXTERNAL	STRUCTURAL	CLEANUP	FINANCIAL	OPPORTUNITY
RESIDENTIAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Laundries, Caravans, Sheds, Tools, Gardens, Fences	Physical Damage to Buildings: Gyprock, Cupboards, Scour of Footings, Houses becoming buoyant (floating off footings)	Clean Carpets, Walls, Clothes; Re-instate Furniture; Remove Mud and Debris	Loss of wages, Living costs (temporary accomodation and food), Time to repair/replace damaged items	Not Applicable
RURAL	Contents of Buildings: Clothes, Carpets, Furniture, Valuables, Fittings, Appliances	External Items: Vehicles, Sheds (stables/barns), Machinery, Tools, Fences, Feed storage, Saddles, Crops &/or Stock, Irrigation Systems	Physical Damage to Structures: Damage to Homestead, Sheds, Access tracks, Protection levees	Clean Homestead and Out-buildings; Remove Debris; Dispose of affected crops &/or stock	Loss of Farm Production and Income, Re-instatement of Pastures, Supplementary feeding of stock (by hand or outside agistment), Stock movement/ transport, Living [illegible]	Sowing or harvesting of Crops, Sale of Stock (at depreciated value or dependent on market influences)
COMMERCIAL	Contents of Buildings: Products, Stock, Fittings, Tools, Machinery, Raw Materials	External Items: Vehicles, Machinery, Display, Raw Materials/Stockpiles, Fences	Physical Damage to Buildings	Dispose of damaged products, stock, materials; Cleaning and Re-instatement	Loss of Productivity and Income, Bank Interest Charges	Loss of existing &/or Potential Trade
PUBLIC AUTHORITIES	Contents of Public Buildings and Facilities	Public Property and Facilities: Parks, Signs, Machinery, Equipment	Physical Damage to Infrastructure: Electricity, Water, Telephone, Gas, Road & Rail Transport Links	Remove Mud & Debris from Facilities, Public & Private Property Repairs (temporary & permanent)	Disruption of Services, Community Service Relief Grants	Provision of Public Service

Public Utilities
There will be disruption to electrical and telephone services in large storm events, however this may occur as a result of lightning or rainfall rather than elevated ocean water levels alone.

Underground water supply, stormwater and sewerage lines could be disrupted due to loss or collapse of road surface, scour of the pipelines or, perhaps, inundation of chambers.

Inundation may release untreated sewage which is also likely to back up inside the buildings or in yards (pits).

Recreational Facilities
A minor part of the CBD carries areas used for active and passive recreation. Damage to public amenities and grassed surfaces is expected in the case of revetment overtopping.

Clean Up Costs
Following a large ocean storm event all public authorities are likely to be involved in the post inundation repair process.

Inundation damages can be defined as being *tangible* or *intangible* and a schematic breakdown of the damages categories is provided in the main body of the text. Tangible damages are those for which a monetary value can be assigned, in contrast to intangible damages, which cannot easily be attributed a monetary value.

B1.3 Tangible Damages

Tangible damages can be sub-divided into *direct* damages, which occur due to physical contact with inundation waters, and *indirect* damages which occur as a result of the disruption of business, trade and other activities. Direct and indirect damages may be referred to as *Potential* or *Actual* damages. Potential damages are the assumed damages if no damage reduction measures are employed and are thus greater than the actual damages. The ratio of actual to potential damages depends upon a number of factors including:
- magnitude of the flood,
- prior flood experience of the community,
- length of warning time.

Direct Damages
Direct damages can be sub-divided between the rural and urban sector. Under direct urban damages there are three broad categories: *Residential, Commercial* and *Public Sector*.

The direct damages under these categories can be grouped under the following headings:
- *Internal* - building contents,
- *Structural* - structure and building fabric,
- *External* - yard, garage, vehicle and other machinery (air conditioning).

Damages to commercial and industrial buildings are much more difficult to quantify for two reasons:

- damages to a given property vary much more than with houses, as they are heavily influenced by the type of business being carried out and the amount of stock carried. This will also vary over time as different businesses use the building,
- industrial enterprises in particular cannot simply be averaged out. Where large factories or warehouses are involved, the only way to get a good estimate of potential damages is to do a site specific survey of the enterprise.

As inundation damages can vary greatly between areas depending upon the type of buildings and contents, an average damages figure is estimated for each of the above categories (residential, commercial and public sector) following an inundation event. This is generally presented as a inundation depth versus inundation damages function. For residential buildings, the size, building fabric, condition of the house and whether it is single or two storey are also taken into account.

Public sector (non-building) damages include:

- recreational/tourist facilities,
- water and sewerage supply,
- telephone supply,
- electricity supply including transmission poles/lines, sub-stations and underground cables,
- roads and footpaths including traffic lights/signs,
- costs to employ the emergency services.

Damages to the public sector can contribute a significant proportion of the total flood costs. In the Inverell flood of February 1991, direct costs to the local Council accounted for 10% of the total direct damages. A single item such as a sub-station may account for a large proportion of the damages bill in a particular flood.

Indirect Damages

Indirect damages are more difficult to quantify. They can be sub-divided into three broad cost categories:

- *Clean-up* - clean carpets, furniture, refrigerator, etc. It also includes the cost of alternative accommodation,
- *Financial* - loss of wages, loss of trade for the commercial/industrial sector,
- *Opportunity* - non-provision of public services.

In a particular locality it would require an extensive survey to evaluate the costs of lost working hours, disruption to business and trade. Nevertheless an indication of the damages can be obtained from previous studies. Generally the indirect damages have been expressed as a percentage of the direct damages. The figure varies greatly depending upon a number of factors including:

- magnitude of inundation,
- time away from home/work,
- category (residential, commercial, industrial).

An average percentage (indirect as a percentage of direct) from a number of post flood surveys is:

- Residential - 15%,
- Commercial - 30%,
- Industrial - 50%.

B1.4 Intangible Damages

Intangible damages are those flood damages which by their nature are difficult to quantify in monetary terms. An example of a *direct* intangible damage is the "loss of visual quality" of an area or "loss of a heritage item". Most intangible damages are *indirect* and commonly occur after the inundation has subsided.

Intangible damages can be categorised as follows:

Residential

Post inundation damages surveys (References B1, B2 and B3) have linked inundation to stress, ill-health and trauma in the residents. For example the loss of memorabilia, pets, insurance papers, etc., may cause stress and subsequent ill-health. In addition, inundation may affect personal relationships by contributing to marriage breakdowns and lead to stress in domestic/work situations. Residents may worry each time there is a threat of inundation. This may be reflected in increased sickness or depression requiring psychiatric help. These effects can induce a lowering in the quality of life of the inundation victims.

Inundation victims may also suffer injuries during an inundation event or during the clean-up process. Whilst the direct costs of the injuries may be accounted for in the inundation damages survey, the physiological effect or discomfort may last for a long time.

The most extreme “intangible damage” that can arise from inundation is death, and unfortunately this is not a rare occurrence. There are many examples of deaths of local residents and rescue workers during inundation events.

Commercial/Industrial/Rural

Whilst a large number of businesses carry insurance for loss of trade during and following inundation until the clean-up is complete, they may still suffer a financial loss. For example the confidence in the business of regular clients may be reduced permanently. Clients may take their business elsewhere during the inundation/clean-up period and may never revert to the original supplier.

Services

The loss of services to customers, e.g., transport disruption, loss of education, loss of power, etc., occur as a result of inundation and these are generally not costed within the tangible damages category.

Environmental

Environmental damage may occur as a result of inundation, for example flora and fauna may be lost. In the short term there may be a deterioration in water quality or vegetation, which may recover in the long term.

Probably the most significant potential environmental impact is the release of pollutants as a result of inundation. Generally this is as a result of inundation of commercial/industrial establishments.

In summary, there is a comprehensive body of available literature on intangible damages which provides many examples. However the costing of such damages in dollar terms is often not possible. These "costs" must not be ignored when determining management options. The literature suggests that the value of intangible damages may equal or exceed tangible damages. It is therefore often necessary to imply a value to the intangible damages to achieve a proper appreciation of proposed works and measures.

B2. ASSESSMENT OF INUNDATION DAMAGES

B2.1 General

Quantification of inundation damages is generally based upon post-flood damage surveys. An alternative procedure is to undertake a self-assessment survey of the inundation liable residents. More recent information will become available from the November 1996 flood at Coffs Harbour. A listing of the most widely known post flood damage surveys is shown in Table B2.

Table B2: Residential Flood Damage Surveys

Location	Year of Flood	Comments
Brisbane	1974	400 residential properties.
Lismore	1974	100 properties. The data were obtained several years after the last major flood.
Forbes	1974	35 properties. The data were obtained several years after the latest major flood.
Sydney (Georges River)	1986	96 properties (2 studies undertaken)
Nyngan	1990	24 residential, 14 commercial and 6 public properties, 4-5 weeks after the flood.
Inverell	1991	4 residential, 20 commercial and 10 public properties, 2-3 weeks after the flood.

The most comprehensive surveys are those carried out for Sydney (Georges River), Nyngan and Inverell. Some of the problems in applying data from these studies to other areas can be summarised as follows:

- varying building construction methods, e.g. slab on ground, pier, brick, timber,
- different average age of the buildings in the area,
- the quality of buildings may differ greatly,
- inflation must be taken in account,
- different fixtures within buildings, e.g. air-conditioning units,

- change in internal fit out of buildings over the years or in different areas, e.g. more carpets and less linoleum or change in kitchen/bathroom cupboard material,
- external (yard) damages can vary greatly. For example in some areas vehicles can be readily moved whilst in other areas it is not possible,
- different approaches in assessing inundation damages. Are the damages assessed on a "replacement" or a "repair and reinstate where possible" basis? Some surveys include structural damage within internal damage whilst others do not,
- varying warning times between communities means that the potential to actual damage ratio may change,
- variations in inundation awareness of the community.

B2.2 Tangible Damages - Residential Properties

Tangible direct damages are generally calculated under the following components:
- Internal,
- Structural,
- External.

Tangible indirect damages can be subdivided into the following groups:
- accommodation and living expenses,
- loss of income,
- clean up activities.

All estimates are actual damages rather than potential damages.

B2.2.1 Direct Internal Damages

Water Studies

In the Water Studies approach internal damages are based upon the following formulae provided in Reference B1.

$$\frac{D}{D_2} = 0.06 + 1.42H - 0.61H^2 \qquad \text{for } H < 1.0\text{m}$$

$$\frac{D}{D_2} = 0.75 + 0.12H \qquad \text{for } H > 1.0\text{m}$$

where,

H	=	height of flooding above floor level (m)
D	=	damage at height (H) above floor level
D_2	=	damage at height of 2m above floor level

At Nyngan and Inverell D_2 was \$12 500 for small houses and \$14 500 for medium/large houses. These values are for 1991 prices. The reference states that *"Damages to individual properties scatter widely around the relationship, which can only be used to reliably estimate the aggregated* *damage to a collection of flood prone dwellings and not the damage to a single dwelling".* Structural damages are not included in the above figures.

CRES

In the CRES approach (Reference B3) internal and structural damages are combined. Data are provided for three groups of buildings, namely Poor, Medium and Good. The data are shown in 1986 prices in Table B3.

Table B3: Residential Damage to Structure and Contents ($1986's)
(Taken from the Georges River Study: Reference B3 - Table B2.2.7)

Overfloor Depth	Poor	Medium	Good	Average
0.0m	$370	$1045	$ 2400	$1270
0.1m	$740	$2090	$4799	$2540
0.6m	$3012	$5713	$10360	$6360
1.5m	$7102	$7595	$13190	$9300
1.8m	$7210	$7711	$13391	$9440

B2.2.2 Direct Structural Damages

In the CRES approach internal and structural damages are combined. In the Water Studies approach structural damage was adopted as approximately $5 000 at both Nyngan and Inverell.

B2.2.3 Direct External Damages

The majority of external damages is attributable to vehicles. However there is a high likelihood that a significant percentage of the vehicles can be moved to high ground even with minimal warning.

At Nyngan external damages were estimated as $4 500, mostly for vehicles, and at Inverell at $2 500 of which $1 500 was for vehicles. In the Sydney 1986 data obtained by CRES an external damages figure of $600 was adopted per property experiencing over ground flooding. In addition a sum of $2 000 per property experiencing over ground flooding in excess of 0.6m was included.

B2.2.4 Indirect Damages

In the Inverell study the indirect damages were taken as $200 for accommodation, $100 for loss of income and $2 100 for clean up activities. The total indirect damages ($2 400) therefore, represented approximately 20% of the direct damages. At Nyngan indirect damages were high due to the extended period residents were away from their homes and were estimated at $7 700 per dwelling flooded above floor level. In this case the indirect damages amounted to approximately 40% of the direct damages. CRES adopted a figure for indirect damages of 15% of the direct damages (Georges River Study).

B2.3 Adopted Tangible Damages - Residential Properties

The adopted values used in this study are provided in Table B4 and Table B5.

Table B4: Residential Depth/Inundation Damage Data ($1998)

Inundation Relative to Floor Level (m)	Internal Damages	Structural & Indirect Damages	External Damages	Total
-0.5	$0	$0	$0	$0
-0.3	$0	$0	$400	$400
0.0	$0	$0	$1500	$1500
0.1	$3918	$2200	$1500	$7618
0.3	$8622	$6600	$1500	$16722
0.5	$12350	$11000	$1500	$24850
1.0	$17400	$11000	$1500	$29900
1.5	$18600	$11000	$1500	$31100
2.0	$19800	$11000	$1500	$32300

Table B5: Residential Depth/Wave Impact Damage Data ($1998)

Inundation Relative to Floor Level (m)	Internal Damages	Structural & Indirect Damages	External Damages	Total
-0.5	$0	$0	$0	$0
-0.3	$0	$0	$400	$400
0.0	$0	$0	$1500	$1500
0.1	$3918	$11000	$1500	$16418
0.3	$8622	$33000	$1500	$43122
0.5	$12350	$55000	$1500	$68850
1.0	$17400	$55000	$1500	$73900
1.5	$18600	$55000	$1500	$75100
2.0	$19800	$55000	$1500	$76300

B2.3.1 Direct Internal Damages

The Water Studies approach to the determination of internal damages was adopted for use in this study. It was decided to adopt a single D_2 value of $20 000 for all residential buildings.

B2.3.2 Direct Structural Damages

Structural damages were assumed to be a linear relationship of $0 at 0 m to $8 000 at 0.5 m. Above this value it was considered that there would be no additional structural damages. In the case of a residential dwelling along the foreshore subject to wave impact, structural damages obtained from the linear relationship were increased by a factor of 5.

It is likely that some buildings subject to wave impact may collapse or have to be destroyed. The cost of this damage has not been included in the analysis.

B2.3.3 Direct External Damages

External damages (laundry/garage) was assumed to be a linear relationship from $0 at 0 m above ground level to $1 000 at 0.5 m. Vehicle damages were assumed to be $0 at 0.2 m and to increase linearly to $500 at 0.5 m above ground level.

B2.3.4 Indirect Damages

Indirect damages were assumed to be a linear relationship from $0 at 0 m to a maximum of $3 000 at 0.5 m. In the case of a residential dwelling along the foreshore subject to wave impact, indirect damages obtained from the linear relationship were increased by a factor of 5. This is to take account of additional clean-up costs as well as the potential for increased accommodation requirements resulting from significant structural damage.

B2.4 Tangible Damages - Commercial Properties

Damages to commercial properties cannot be estimated as accurately as damages to residential properties for a number of reasons, including:

- less post-flood surveys have been undertaken in Australia,
- some commercial properties are insured against flood loss, if this is the case the insurance premiums need to be considered in assessing flood damages,
- flood damages can vary greatly from building to building. For example an electrical retail shop may suffer more damages than say a sandwich shop, as the latter has less high value stock. On the other hand there is more opportunity to reduce this actual damage in the former as the items can be easily moved by staff if there is sufficient warning and awareness. In large premises the flood damages depends on the care taken in moving stock. Carpets are high value items and cannot be easily moved whilst the cars in a car showroom can be easily moved. In many floods there is no safe place to put the cars, yet carpets can be stacked on each other or raised,
- the damages can vary from year to year as the usage of a particular premises changes. Damages may also vary on a seasonal or weekly basis depending upon the type of business,
- indirect damages (loss of trade) may be significant and this is difficult to estimate.

In this study tangible direct commercial damages were estimated using data taken from Reference B1, where:

$$D = ‹ \log_e (H\text{-}B) + y$$

where, D = unit damage (\$ per m^2)
H = depth of inundation above floor level (m), and
‹ , B and y are parameters determined from field survey. The following parameters were adopted for use in this study: Commercial ‹ = 14.6, B = 0.19, y = 86.9.

Note: The above value of D (\$ per m^2) relates to 1991 prices. An inflation factor of 1.5 was used to convert D to 1998 prices.

Commercial floor areas within the CBD were assumed to be 150 m^2 unless aerial photography indicated significantly otherwise. Indirect tangible damages were taken as 100% of direct damages. This figure includes external damages, structural damages, financial loss and clean up costs. In the case of a commercial building along the foreshore subject to wave impact, the unit damage (\$ per m^2) was increased by a factor of 5 to take account of additional structural damage and cleanup costs.

B2.5 Tangible Damages - Public Infrastructure

The damages to public infrastructure and utilities (excluding buildings which are taken as commercial properties) include:
- training wall and walkway,
- water, stormwater and sewerage lines,
- telecommunications,
- road infrastructure,
- parks and street gardens,
- other public assets.

Little data are available for establishing costs to public infrastructure, and the data from Nyngan and Inverell show that it varied from 17% to 36% of the total damages bill. In this study damages to public utilities were assumed to be somewhat high as a result of wave impacts and saline water damage to the foreshore and utilities such as roads, gardens and telephone lines. A linear relationship from \$0 at 0 m to \$1 500 000 at 0.5 m.

It should be emphasised that these figures include only tangible (direct or indirect) damages to assets, the cost of intangible damages has not been evaluated. Available literature suggests that the extent of intangible damages may equal or exceed the tangible damages.

B2.6 Average Annual Damages

While the total damage figure in a given event is useful to get a "feel" for the magnitude of the inundation problem, it is of little value for economic evaluation. When considering the economic effectiveness of a proposed mitigation option the key factor is the total damage prevented over the life of the option. This is a function not only of the high damage which occurs in large events but also of lesser (but more frequent) damage which occur in small events.

The standard way of expressing inundation damage is in terms of *Average Annual Damages* (AAD). These are calculated by multiplying the damage that can occur in a given event by the probability of the event occurring in a given year. These numbers are then summed across the range of events. By this means the smaller, more frequent events are given a greater weighting than the rare, catastrophic events.

The AAD for the Batemans Bay CBD study area was derived from an examination of the 5%, 2% and 1% events. On this basis the AAD for the area was estimated to be approximately $110 000 up to the 1% event, excluding intangible damages.

The AAD can then be converted into a Present Day value by adopting an appropriate discount rate and time period. Assuming a 50 year planning period and adopting a 7% discount rate results in a Present Day damages total of $1 500 000 for the CBD area not including intangible damages. Note, a lower discount rate would produce a higher damages total.

B3. REFERENCES

B1. NSW Department of Water Resources
Inverell Flood Damage Survey February 1991 Flood
Water Studies Pty Ltd - November 1991.

B2. NSW Department of Water Resources
Nyngan 1990 Flood Investigation - Chapter 9
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B3. Public Works, Department of Water Resources
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Centre for Resource and Environmental Studies, Australian National University, and Environmental Management Pty Ltd Sydney - September 1990.

APPENDIX C: COUNCIL'S POLICIES ON FLOOD LIABLE DEVELOPMENT

APPENDIX D: WHARF ROAD FORESHORE ALIGNMENT ASSESSMENT

APPENDIX D: WHARF ROAD FORESHORE ALIGNMENT ASSESSMENT

D1. SEDIMENT MOVEMENT PROCESSES

Sediment movement processes for the Wharf Road/channel margin shoal area are dominated by major flood/non-flood cycles.

During major floods a secondary flood channel is scoured along the Wharf Road foreshore and the sands deposited offshore into the region between Surfside Beach and Square Head. After the floods the sands are reworked back towards the shore by waves, rebuilding the channel margin shoal and eventually Wharf Road beach prior to erosion during the next major flood.

The Estuary Processes Study (WBM, 1999) described two fundamental foreshore configurations for the Wharf Road area:

- a wave dominated shoreline where sand from the channel margin shoal moves into the area and forms a sand spit attached to the foreshore in a direction more or less parallel to the predominant ocean wave crests,
- a nearshore current dominated foreshore shape where the foreshore (usually bordered by a secondary tide channel), runs more or less east-west from Surfside Beach through to the river.

The accretion which occurred in the Wharf Road area through the 1980's and 90's is part of a wave dominated period when sand moved on to the Wharf Road foreshore. However, indications from historical records are that the current dominated eroded foreshore shape is a more common/likely condition, separated by shorter periods with a wave dominated foreshore shape.

During wave dominated foreshore periods large quantities of sand move on to the Wharf Road foreshore. During current dominated foreshore periods both accretion and erosion can occur along Wharf Road but at a significantly slower rate.

D2. ALIGNMENT ASSESSMENT

Based on the available information sediment movement in the Wharf Road area should continue to be dominated by flood and non-flood cycles. On this basis, the substantial accretion which has occurred in the area throughout the 1980's and 1990's could continue until the beach is higher and more extensive. Alternatively, major flooding in the Clyde River could again scour the deposited sand and recommence the cycle.

To date, the accretion and erosion cycle has not been a significant coastal hazard in terms of property damage because the area is largely undeveloped sand flats the loss of which simply changes the local beach alignment and provides a sediment supply for other areas of the bay. The low key trapping of sand which has been undertaken may have accelerated natural accretion, but would not prevent major flood scour. However, more permanent trapping of sand or obstructions to flows such as construction of a seawall could change the flood/non-flood cycle and may impact on the sediment supply to Surfside and Cullendulla Beaches.

To help assess the likely consequences of constructing a physical barrier along the Wharf Road foreshore WBM (2000) examined historical shoreline positions in the Wharf Road area between 1898 and 1999. The assessment procedure was based on four hydrographic charts (1898, 1922, 1931 and 1978) and eight aerial photographs (1949, 1964, 1977, 1981, 1982, 1986, 1987, and 1999). Based on this assessment WBM estimated the percentage location of the high tide boundary and how often a range of different wall profiles would be exposed to direct flows. This was used as an indicator of the time a beach would be located in front of various wall alignments.

The above (WBM, 2000) assessment procedure has a number of inadequacies. One particular problem is the heavy (50%) weighting given to the ten year period between 1977 and 1987 when the foreshore alignment shape was wave dominated and not typical of past conditions. Another is the use of four hydrographic charts which gives these a one third weighting. These charts potentially introduce substantial errors because the charts are predominantly indicators of channel depths for navigation. The foreshore information was usually not physically surveyed and was generally added for visual reference only.

To address some of these problems the percentage location of the foreshore was recalculated with a weighting based on the number of years between assessment dates (see Table D1). This was easily established from the progressive change in foreshore shape over time. Similar dates were used to the WBM (2000) assessment but without the 1978 hydrographic chart and with additional reference to 1962, 1980, 1984 and 1996 aerial photography.

Table D1: Wharf Road Foreshore Alignment - Year Multipliers

Year	Multiplier		
	Method 1	Method 2	Method 3
1894	12	6	6
1922	16	8	8
1931	13	6.5	6.5
1949	16.5	12	12
1964	14	14	28
1977	11.5	11.5	23
1987	11	11	22
1999	6	6	12
Total	**100**	**75**	**117.5**

Three different methods were adopted:

- **Method 1** used a weighting based on the time periods between assessment dates,
- **Method 2** used a weighting based on half the time period for hydrographic charts to reflect their uncertainty,
- **Method 3** also used half for the hydrographic charts, but the weighting was double for the period since the bridge and the existing Wharf Road seawall were constructed to reflect the changed hydrodynamic conditions.

The analysis involved the following steps:

- The shoreline location was noted for each date and placed on the grid created in WBM (2000). The definition of onshore land was the high tide watermark on the charts and as indicated by the boundary of wet sand in the photographs.

- Plots were produced (similar to WBM (2000) Figure D1) showing the percentage of time for each onshore land location. Separate plots were produced for each of the three weighting methods adopted for this assessment.

- The same grid was placed over the 1996 aerial photograph (Figure D2) and the value of each nodal point, as taken from the plots, placed on the grid. Interpolating between each of the nodal points, the 80%, 60% and 40% lines of shoreline location were drawn in Methods 1, 2 and 3.

It should be noted that the 100% shoreline location was considered to be the highest tidal watermark shown on the photographs and charts used, in this case primarily the 1964 shoreline.

D3. FINDINGS

The revised assessment procedure produced significantly different percentage alignments to those produced by WBM (2000), see Figures D3, D4 and D5. For the purposes of comparison the 60% lines for Methods 1, 2 and 3 as well as from WBM (2000) were placed together on a separate aerial photograph (see Figure D6).

Based on the available information, a constructed physical barrier on the landward side of the 100% line is unlikely to be exposed and so is unlikely to have any adverse impacts on sediment movement. A barrier along the 40%, 60% or even the 80% line is likely to be exposed at some stage, and so will have some impact on sediment movement. The nature and extent of these impacts is very difficult to evaluate.

WBM (2000) have assessed that the impacts of a barrier along the eastern Wharf Road foreshore will be minimal and that even when theoretically exposed the barrier could have a sand beach which would allow access along the foreshore. This assessment may be valid, however, it is also possible that a major flood event would scour a deep efficient channel along the fixed foreshore which would be more permanent than existing/past channels.

Such a channel may increase the time in which there is no beach along the Wharf Road foreshore. Such a channel may also increase or decrease the volume of sand delivered to Surfside Beach, which may in turn result in sand moving from the beach on to and along the Wharf Road foreshore.

Because of the complexity of the shoals and the variability of the wave and current climate in Batemans Bay it will not be possible, even with extensive numerical or physical modelling to determine which foreshore scenario will develop. Therefore, in accordance with the principles of ecologically sustainable development, the 100% line has been adopted for this Management Plan.