BRISBANE WATER FORESHORE PROTECTION
INVESTIGATION AND DESIGN

NSW Public Works
Manly Hydraulics Laboratory

Report MHL638
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Foreword

Manly Hydraulics Laboratory (MHL) was commissioned by Gosford City Council (GCC) to carry out the necessary investigations and design of suitable foreshore protection works at nominated sites around Brisbane Water.

MHL personnel involved with this project included:
- W Strachan - Project Manager
- J Murtagh - Project Engineer
- A Gordon - Technical Advisor

GCC personnel involved with this project included:
- M Alsopp - Program Manager - Environment
- C Argaet - Recreation Section Technical Officer
- R Hall - Technical Officer

GCC also engaged Coffey Partners International Pty Ltd (Coffey) to advise on geotechnical aspects relating to the investigation, selection and design of foreshore treatments.

It should be noted that a draft report on this project was issued to GCC in April 1993 and Council drew upon this draft report for construction purposes. This final publication was printed in March 1995 and is, in the main, unchanged from the April 1993 draft.
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1. Introduction

1.1 Background

In 1992, Gosford City Council (GCC) resolved to carry out foreshore protection works at several reserves with frontages to Brisbane Water. Funding for the works was available from the Federal Government under the One Nation Program of Works, the State Government under the State Estuary Management Program and also from the GCC's Capital Works budget. A total budget of $18 million was allocated for the proposed works under the control of the GCC Recreation Section.

All of the foreshore locations have experienced a varying degree of erosion. Some sections have had limited physical attempts to mitigate the erosion. GCC resolved the following priorities for these works:

1. Woy Woy (Brickwharf Road, Lions Park, North Burge Road)
2. Empire Bay (including finishing a small foreshore section near Pine Avenue, Davistown)
3. Gosford Waterfront
4. Saratoga Oval, northern foreshore
5. Brisbane Water Drive (southern end of Tascott Straight)
6. Yattalunga

1.2 Objectives

Manly Hydraulics Laboratory (MHL) was engaged by GCC to carry out the investigation and design of suitable foreshore protection works for each of these sites. The aims of GCC's brief to MHL were given as follows:

- To provide GCC with pre-construction technical advice as to the optimum foreshore protection method for each location,
- To advise upon design criteria, such as crest and toe levels, protection unit type and size, alignment and so forth,
- To provide ongoing technical advice during the construction period

1.3 Scope of the Project

GCC undertook to supply MHL with survey information of typical cross-sections for each location, and to conduct a detailed site inspection with MHL staff to assess and discuss both the environmental aspects and local community concerns regarding each site.

GCC advised that it had a ready supply of good quality sandstone rock, at a very competitive price, and wherever possible this was to be the material used at those sites requiring hard protection. In considering the protective measures to be adopted, GCC advised on the land...
use and water access requirements for each particular section of foreshore and its future requirements. Where possible GCC also wanted to "standardise" the design of the seawall design from one site to the next.

As the designs were undertaken, with input of survey information (by GCC) and geotechnical assessment (by Coffey) advance designs were forwarded to enable GCC to schedule its day labour resources.

This report encompasses the designs for all the foreshore sites. Each site is described in terms of its "Present Condition" prior to the recommended foreshore treatment, an "Outline of the Options" suitable for the site, and a description in text and diagrams of the "Recommended Foreshore Treatment", the latter having been discussed and agreed with GCC's representatives.

The sites are shown in Figure 1-1 and reported on in the following sections of this report in the following order:

- Section 2 North Burge Road Reserve, Woy Woy
- Section 3 Lions Park, Woy Woy
- Section 4 Brickwharf Road, Woy Woy
- Section 5 Davistown Eastern End
- Section 6 Empire Bay - North West of Boatshed
- Section 7 Empire Bay, East
- Section 8 Gosford Waterfront
- Section 9 Saratoga
- Section 10 Koolewong
- Section 11 Tascott
- Section 12 Yattalunga
2. Woy Woy, North Burge Road Reserve

2.1 Present Condition

The North Burge Road Reserve borders the eastern side of Woy Woy and is the western foreshore of the Rip Channel. From Lions Park, Woy Woy at its northern end to Bowden Road at the southern end of the proposed works, it is a distance of 930m. Along this length the width of the reserve, from the adjoining property boundaries to the existing foreshore escarpment, varies between approximately 10m and 30m. The general level of the grassed terrace of the reserve is at approximately 0.91m AHD to 1.0m AHD and consequently would be overtopped/ inundated by extreme water levels. The design wave conditions for this site would be generated by boat wake and could be in the order of 0.2m to 0.3m.

A number of foreshore treatments such as tipped rubble revetments, timber retaining walls and mortared masonry retaining walls have been installed on an ad-hoc basis along the foreshore by the owners of properties adjoining the reserve. These structures are scattered intermittently along the foreshore displaying a number of different alignments which represent the alignment of that particular section of foreshore at the time of construction. With the exception of the remnant of the brick wharf and a mortared masonry wall immediately south of it, the majority of these structures are in advanced stages of disintegration displaying significant scouring behind them. (Plate 1 shows typical sections of this foreshore).

The erosion escarpment along the foreshore varies somewhat in height but is generally of the order of 0.7m in height. Extending out from the toe of the escarpment is a sand covered intertidal berm. At the northern end this berm is about 30m wide with a grade of 1:30. From about half way south to the vicinity of Park Road the berm widens and flattens. South of Park Road to Bowden Road the intertidal berm is about 160m wide with a slope of around 1:200. Beyond the intertidal berm the bed drops off rapidly into the main channel.

Coffey Partners International Pty Ltd (Coffey) were retained by GCC to conduct sub-surface investigations at the site consisting of excavating, logging and sampling nine hand auger holes of varying depths at approximately 50m intervals and conducting 16 dynamic cone penetrometer tests at about 25m intervals. Based on their investigations Coffey concluded

"The generalised sub-surface profile at the Woy Woy site is summarised as follows.

TOPSOIL/FILL: (CL) Sandy Clay, low plasticity, brown, some silt, wet, observed unit thickness varied from 0 to 0.6m, overlying

ALLUVIUM: (SP) Sand, fine to medium grained, grey and brown, some sand and sandy clay zones, loose to dense generally medium dense, unit observed to depths of 1.8m."

Coffey assessed that the allowable bearing capacity at the North Burge Road site is 100kPa.
2.2 Outline of Options

While there is a range of options for foreshore treatment of this site GCC's preferred treatment is a dry jointed sandstone block wall, with the sandstone rough cut and keyed in a random pattern. The wall should extend from the eastern boat ramp at Lions Park, Woy Woy southward, generally along the existing escarpment alignment to Bowden Road.

2.3 Recommended Foreshore Treatment

Figure 2-1 shows a typical cross-section of the recommended foreshore treatment for this site. This diagram needs to be read in conjunction with this text. It should be noted that this design is for a typical cross-section and may require appropriate on-site modification which should be directed and supervised by experienced GCC staff.

The recommended treatment at this site is a sandstone block seawall, comprising large rough-cut sandstone blocks varying in size from say 0.5 tonne to 3.0 tonne. The blocks are to be keyed in a random pattern and dry grouted. The larger blocks should be placed at the base with weep holes provided by spacing blocks with a 20 to 50 mm gap. It is proposed to shape some of the sandstone rocks so as to provide the 10 mm wide footing required.

The seawall toe should be trenched into the sand to a depth of -0.80 m AHD, with the rear face of the excavation trimmed up to the level of the terrace as shown in Figure 2-1. A gravel under-footing 0.3 m thick encapsulated in geotextile fabric should be placed in this trench to provide load spreading at the joints between boulders in the footing course and minimise differential settlements. This will bring it up to the footing level of -0.5 m AHD to ensure 0.5 m embedment while allowing for scour at the toe of the wall.

The crest has been set at 1.00 m AHD which is marginally above the level of the grassed terrace and is about the level of the design (still) water level with an annual recurrence interval of 1 in 5 years. During elevated water level conditions the seawall and the grassed terrace will certainly be overtopped by wave action superimposed on the elevated water level conditions. GCC should be aware that this will scour and damage the grassed area behind the seawall from time to time, necessitating some level of maintenance. The design wave conditions for this site would be generated by boat wake and could be in the order of 0.2 to 0.3 metres.

In relation to this wall design Coffey's report (refer Appendix A) makes the following observations: "It is understood that the proposed wall is to be constructed of rough cut sandstone and railway ballast. It is further understood from discussions with the wall designers that the maximum wall height is to be approximately 1.5 m and it is expected that the wall will be occasionally overtopped. It is anticipated that the wall will have an embedment of approximately 0.5 m with a height of about 1 m.

Given the proposed wall heights and the anticipated unit weights of the construction materials, it is assessed that the required allowable bearing pressure to support the wall is approximately 40 kPa."

The recommended treatment at the back of the wall is shown for a typical cross-section in Figure 2-1. Geotextile fabric should be placed under the base of the seawall encapsulating the underfooting and return up the landward side of the sandstone block wall. The cross-section area behind the seawall for a distance of say 1500 mm at the surface should be backfilled with...
suitable graded material. The surface should be grassed with salt-water resistant grass species.

The overall drainage from North Burge Road Reserve needs to be assessed and accommodated by GCC in association with this seawall design.

The alignment of the seawall needs to be determined on site but in general should follow the line of the existing foreshore escarpment. The seawall should terminate in the vicinity of Bowden Road at a location to be determined on site. At this point the wall should be returned for a short distance (say 2 metres) into the reserve terrace if no existing suitably armoured foreshore feature is available for use as a terminus.

It is anticipated that there will be some loss of sand from the slope in front of the (near) vertical seawall, due to the higher reflected wave energy. An allowance for this anticipated loss has been made in determining the levels of both the footing and gravel underfooting of the wall.
Much of the foreshore is lined with rubble (looking south)

Some sections are more substantial (south of brick wall)
I
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AH
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Seawall to comprise of large
rough cut sandstone blocks
(0.5 tonne to 3 tonne) keyed
in random pattern and dry
grouted

Seawall crest at 100 AHD

Larger blocks used for bottom row of
seawall with weep holes provided by
spacing blocks with 20 to 50mm gaps

Geotextile fabric return
up inside of wall and
encapsulating the gravel
underfooting

Geotextile fabric retum
up inside of wall and
encapsulating the gravel
underfooting

NOTES
1 This design profile is typical for this foreshore section and may
require appropriate on-site modification
2 Figure 2-1 needs to be read in conjunction with the text for
North Burge Road Reserve
3 As indicated in Coffey and Partners Report No G0625/1-AB
the footing should be 1.0m wide with embedment of at least 0.5m
4 Council accepts that crest of this seawall will occasionally be
overtopped
5 Gravel underfooting for load spreading to minimise differential
settlement at joints between blocks in the bottom course

Graded backfill

Surface area behind seawall grassed

Existing terrace
100 AHD

Existing rocky / sandy slope

Sandstone footing at -0.50 AHD or deeper

Seawall toe trenched into sandy bed

Gravel underfooting at -0.80 AHD or deeper

Minimum thickness of gravel underfooting 300mm

Figure 2-1

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Report No 638

NORTH BURGE ROAD RESERVE - WOY WOY
3. Lions Park - Woy Woy

3.1 Present Condition

The Lions Park foreshore is at the tip of a peninsula east of Woy Woy. The Woy Woy channel runs east-west along the park's northern boundary, and the main channel of Brisbane Water runs approximately north-south along the eastern boundary of the park.

At each extremity of the park are boat launching ramps built by GCC with PWD funding in the mid 1980s. Other foreshore and park features associated with these boat launching ramps include car and trailer parking, fish cleaning tables, mooring jetties, picnic facilities (built by the Lions Club) and landscaping.

The foreshore from the eastern ramp moving north exhibits littoral erosion, with a small escarpment cutting back into the grassed terrace of the park. A sandy pebble beach slopes out into the water at a grade of about 1:20. Within the tidal zone, shale outcrops break the sandy surface.

Limited investigation of the foundation conditions at this site was carried out by driving a polycarbonate tube at the most easterly end of the point. A 10mm sand layer overlays (what is believed to be) the shale into which no penetration could be achieved by driving the polycarbonate tube with a driver of approximately 25 kg. Visual inspection of the eastern foreshore showed the shale to extend south past the boat ramp. The Woy Woy channel foreshore to the north of the park displayed deeper sand deposits with no shale outcrops visible. Large boulders placed along both foreshores appear to have settled to a stable position and it is not anticipated that foundation conditions will present any particular problems for a fitted rock wall construction at this site.

In an unsuccessful attempt to arrest the erosion, a line of sandstone rocks of varying sizes was placed by GCC in the 1980s. Over the subsequent years, this single row of rocks has been infiltrated and over-washed by wave action with foreshore erosion continuing unabated behind them. Plate 2 visually records the foreshore of Lions Park.

The general level of the grassed terrace of Lions Park is at approximately 0.9 m AHD and consequently would be overtopped/inundated by extreme water levels. The design wave conditions for this site would be generated by boat wake and could be in the order of 0.2 to 0.3 m.

The north-eastern corner of Lions Park is also subject to erosion, which continues around the corner into Woy Woy channel. The degree of foreshore erosion eases for the foreshore approaching the western boat ramp. Indeed this area would seem to be in receipt of some of the sand deposited from littoral drift. In this area, there is also a line of rocks placed along the line of the edge of the grassed terrace. Unlike the eastern boundary, there is no erosion behind the row of rocks and the foreshore appears to be reasonably stable.
There is some minor erosion either side of the western boat ramp due to runoff, indeed the ramp runoff grooves slope to the edge of the ramp rather than the centre.

3.2 Outline of Options

While there is a range of options for foreshore treatment of this site GCC's preferred treatment is a dry jointed sandstone block wall, with the sandstone rough cut and keyed in a random pattern. The wall should extend from the eastern boat ramp around the north-east corner of the park for a distance to be determined on site.

3.3 Recommended Foreshore Treatment

Figure 3-1 shows a typical cross-section of the recommended foreshore treatment for this site. This diagram needs to be read in conjunction with this text. It should be noted that this design is for a typical cross-section and may require appropriate on-site modification which should be directed and supervised by experienced GCC staff.

The recommended treatment at this site is a sandstone block seawall, comprising large rough-cut sandstock blocks varying in size from say 0.5 tonne to 3.0 tonne. The blocks are to be keyed in a random pattern and dry grouted. The larger blocks should be placed at the base with weep holes provided by spacing blocks with a 20 to 50 mm gap. It is proposed to shape some of the existing sandstone rocks already on site and import others.

The seawall toe should be trenched into the shale with this footing cleaned to a solid foundation. The crest has been set at 100 m AHD which is marginally above the level of the grassed terrace and is about the level of the design (still) water level with an annual recurrence interval of 1 in 5 years. During elevated water level conditions the seawall and the grassed terrace will certainly be overtopped by wave action superimposed on the elevated water level conditions. GCC should be aware that this will scour and damage the area behind the seawall. The design wave conditions for this site would be generated by boat wake and could be in the order of 0.2 to 0.3 metres.

The recommended treatment at the back of the wall is shown for a typical cross-section in Figure 3-1. Geotextile fabric should be placed under the base of the seawall and return up the landward side of the sandstone block wall. The cross-section area behind the seawall for a distance of say 1500 mm at the surface should be backfilled with suitable graded material. The surface should be grassed with salt-water resistant grass species.

The overall drainage from Lions Park needs to be assessed and accommodated by GCC in association with this seawall design.

The alignment of the seawall needs to be determined on site but in general should follow the line of the existing row of sandstone rocks. The seawall should terminate along the north side of the park at a location to be determined on site. At this point the wall should be returned for a short distance (say 2 metres) into the reserve terrace.

It is anticipated that there will be some loss of sand from the slope in front of the (near) vertical seawall, due to the higher reflected wave energy.
Previous rock protection ineffectual

Typical foreshore along north side of Lions Park
Seawall to comprise of large rough cut sandstone blocks (0.5 tonne to 3 tonne) keyed in random pattern and dry grouted

Surface area behind seawall grassed

Existing terrace 0.90 AHD

Graded backfill

Geotextile fabric return up inside of wall

Clean footing to solid foundation

Seawall crest at 1.00 AHD

Larger blocks used for bottom row of seawall with weep holes provided by spacing blocks with 20 to 50mm gaps

Existing rocky/sandy slope

Seawall toe trenched into shale (?) Level to be confirmed on site

NOTES

1 This design profile is typical for this foreshore section and may require appropriate on-site modification

2 Figure 3-1 needs to be read in conjunction with the text for the Lions Park Site

3 IMPORTANT - design subject to site investigation by Coffey Partners regarding "solid foundation"

4 Council accepts that crest of this seawall will occasionally be overtopped
4. Brickwharf Road Reserve - Woy Woy

4.1 Present Condition

The Brickwharf Road Reserve borders the northern side of the township of Woy Woy and is on the southern foreshore of Woy Woy Channel between the War Memorial Park to the west and Lions Park to the east. The War Memorial Park is a level filled area surrounded by a brick retaining wall at the toe of which is a cobble beach along the foreshore, which GCC deemed to be stable and excluded from the brief for this study. Lions Park and its foreshore are discussed in Section 3.

In the area immediately to the east of the War Memorial Park the grassed terrace of Brickwharf Road Reserve is about 30m wide with an elevation of around 10m AHD. As the reserve progresses east to a point about 160m past the War Memorial wall the terrace gradually dwindles to a width of about 15m. This area has picnic shelters, seats and mature trees of various species scattered about it, and is a popular family picnic area and fishing spot. Plate 3 shows the foreshore of this area east of the War Memorial Park.

At the waterfront of this section of reserve there is a low escarpment generally less than 300mm high, which is partially undercutting a casuarina toward the eastern end of the terrace. To date only the upper part of the root ball has been exposed so that while the casuarina is under stress it is not in imminent danger of collapse. Below the terrace escarpment a narrow intertidal beach generally about 10m wide with a slope of around 1:10 extends out to the drop off into the main Woy Woy channel. The beach at the top of the drop off is littered with oyster encrusted boulders which are reported to be jettisoned ballast rocks. Further up the slope towards the terrace the beach is sandy, providing a good children’s play area. At both the eastern and western end of this terraced section of the reserve, small clumps of mature mangroves are established on the intertidal beach.

Approximately 550m west of the War Memorial Park wall a 375mm diameter pipeline with no headwall and a broken end emerges onto the beach, with no obvious scour hole at its outlet point. It is not clear whether or not it is blocked or is connected to any inlets upstream. This section of foreshore is quite sheltered with Pelican Island forming the northern foreshore and no significant fetch available for generation of wind waves. Wave activity in this reach of Woy Woy Channel, which in this vicinity is tidal and flows approximately east west, would be generated mainly by boat traffic.

To the east of the terrace area just described, the foreshore reserve rapidly dwindles in width with the foreshore presenting a north eastern exposure for a distance of approximately 50m. In this vicinity the intertidal beach rapidly narrows and steepens toward the south eastern end where two major drainage structures are located.

The more westerly structure is the outlet of a 915mm (3') diameter pipeline with headwall and wingwalls. Scour in this vicinity has exposed the back faces of the headwall and wingwalls.
and some of the pipeline. The pipeline runs basically north south orthogonally under Brickwharf Road and longitudinally under the access road to Woy Woy Oval to the south. It is not clear from a surface inspection whether or not this pipeline currently has any catchment or is a disused remnant.

Some 5m further east a major drainage channel discharges into Woy Woy Channel via twin 1830mm (6') x 1050mm (3' 6'') box culverts, with a concrete headwall and wingwalls but no apron. There is a significant scour hole at the toe of the structure and evidence of scour beside the wingwalls which has previously been ineffectively treated with rubble and mass concrete armouring. The box culvert runs orthogonally under Brickwharf Road to a concrete lined channel which appears to drain both the Woy Woy Oval and areas further south. Both the concrete lined channel and box culverts appear to have been constructed in re-alignment of a pre-existing tidal creek. Plate 3 depicts the culvert outlets and the foreshore to the east.

For a distance of about 100m east of the culverts the foreshore escarpment (in the order of 600mm to 900mm high) continues to follow the northern edge of the road reserve. The intertidal beach along this section is of the order of 20m wide with the Woy Woy Channel alignment having moved a little south toward Brickwharf Road. The intertidal beach replaces the foreshore reserve in this vicinity. Numerous small fishing craft moor along the foreshore in this vicinity.

Further east the Woy Woy Channel moves northward again and the foreshore reserve is once again re-established to the north of the road reserve. The foreshore reserve gradually widens as it progresses east north east toward Lions Park.

4.2 Outline of Options

A number of options have been considered for foreshore protection treatments at this site including re-establishing screening vegetation, beach nourishment, construction of an offshore breakwater, construction of groynes, construction of revetment type walls, construction of near vertical interlocking sandstone rock walls and amplification of drainage works. The site is somewhat varied in terms of both landscape and landuse so that a single treatment is not judged to be universally applicable to all areas of the site. The site has been divided according to functional areas and treatments applied to each area which are in sympathy with the use of that area. The functional areas which have been identified within this site can be summarised as

(i) the recreational grassed terrace and sandy beach at the western end of the site,

(ii) the tidal creek outlet in the middle of the site, and

(iii) the intertidal mooring area at the eastern end of the site

4.3 Recommended Foreshore Treatments

- Foreshore Reserve East of the War Memorial Park

The recommended foreshore treatment for the recreational grassed terrace and sandy beach at the western end of the site is a combination of "soft" foreshore treatments, including limited re-establishment of screening mangroves, limited localised nourishment and landscaped access management of the grassed terrace edge.
Small scale localised sand nourishment of the upper beach profile should be carried out where tree roots have been exposed by erosion, such as those of the casuarina toward the eastern end of the terrace. Sufficient quantities of suitable sand should be available from excavations for the footings of the wall proposed to be built at the eastern end of the site beyond the tidal creek outlet.

Screening mangroves should be re-established along the short section of tidal creek foreshore between the existing mangroves at the north eastern corner of the grassed terrace and the outlet of the 915mm diameter concrete pipeline which crosses Brickwharf Road and discharges into Woy Woy Channel to the east of the grassed terrace. Some temporary screening may be required to protect seedlings from wave attack while they become established. In respect of this revegetation effort the publication "A Guide to Mangrove Transplanting" jointly produced by SPCC, Division of Fisheries and Concord Municipal Council is recommended (copy attached, refer to Appendix B).

- Culvert Outlets
The culvert outlets of the tidal creek should be upgraded. The status of the 915mm diameter pipe culvert should be determined. If the culvert is no longer in use it should be either removed or blocked off and burned. If the culvert is still in use the existing headwall should be removed, the scour hole at the outlet should be filled with granular material to form a working platform and the culvert should be extended by one pipe length. The extended culvert should then have a concrete headwall with wingwalls and apron fitted to the end with rip-rap scour protection placed in front of the apron.

The twin 183m x 105m concrete box culvert should have its existing head and wingwalls removed, the scour hole in front of it should be filled with granular material to form a working platform and the culvert should be extended in length from 2m to 3m (depending on available precast unit lengths). The extended culvert should then have a headwall with wingwalls and apron fitted to the end with rip-rap scour protection placed in front of the apron.

- Foreshore East of the Culverts
Figure 4-1 shows a typical cross-section of the near vertical interlocking sandstone rock seawall which should be constructed along the remainder of the foreshore east of the culverts. This diagram must be read in conjunction with the following text. It should be noted that the design is for a typical cross-section and may require appropriate on-site modification at some points which may be atypical such as in the vicinity of the culvert wingwalls. On-site modifications should be directed and supervised by experienced GCC staff.

The recommended treatment at this site is a sandstone block seawall, comprising large roughly cut sandstock blocks varying in size from say 0.5 tonne to 3.0 tonne. The blocks are to be keyed in a random pattern and dry grouted. The larger blocks should be placed at the base with weep holes provided by spacing blocks with a 20 to 50mm gap. It is proposed to shape some of the sandstone rocks so as to provide the 1.0m wide footing required.

The seawall toe should be trenched into the sand to a depth of -0.80m AHD, with the rear face of the excavation trimmed up to the level of the terrace as shown in Figure 4-1. A gravel under-footing 0.3m thick encapsulated in geotextile fabric should be placed in this trench to provide load spreading at the joints between boulders in the footing course and minimise differential settlements. This will bring it up to the footing level of -0.5m AHD to ensure 0.5m embedment while allowing for scour at the toe of the wall.
The crest has been set at 100m AHD which is marginally below the level of the grassed footway of Brickwharf Road and is about the level of the design (still) water level with an annual recurrence interval of 1 in 5 years. During elevated water level conditions the seawall and the grassed footway will certainly be overtopped by wave action superimposed on the elevated water level conditions. GCC should be aware that this will scour and damage the grassed area behind the seawall from time to time, necessitating some level of maintenance. The design wave conditions for this site would be generated by boat wake and could be in the order of 0.2 to 0.3 metres.

The recommended treatment at the back of the wall is shown for a typical cross-section in Figure 4-1. Geotextile fabric should be placed under the base of the seawall encapsulating the underfooting and return up the landward side of the sandstone block wall. The cross-section area behind the seawall for a distance of say 1500mm at the surface should be backfilled with suitable graded material. The surface should be grassed with salt-water resistant grass species.

The overall drainage from Brickwharf Road needs to be assessed and accommodated by GCC in association with this seawall design.

The alignment of the seawall needs to be determined on site but in general should follow the line of the existing foreshore escarpment taking into account the location of the extended culvert. The eastern end of the seawall should terminate in the vicinity of Station E in GCC's survey dated 5 November 1992 at a location to be determined on site. At this point the wall should be returned for a short distance (say 2 metres) into the reserve terrace if no existing suitably armoured foreshore feature is available for use as a terminus. At the western end the wall should link to the eastern wingwall of the twin 1.83m x 1.05m concrete box culvert. A similar wall should link the western wingwall of the box culvert to the eastern wingwall of the 915mm diameter pipe culvert. Another similar wall should form a return from the western wingwall of the pipe culvert to the grassed terrace so as to fully armour this section of foreshore and prevent failure consequent on outflanking of the wall.

It is anticipated that there will be some loss of sand from the slope in front of the (near) vertical seawall, due to the higher reflected wave energy. An allowance for this anticipated loss has been made in determining the levels of both the footing and gravel underfooting of the wall.

Obviously GCC will need to liaise with the owners of boats moored along this foreshore and accommodate agreed modifications to this design. Access from the seawall crest to the boats may be an issue which could be provided by recessed steps within the seawall or via timber stairs.
Typical foreshore west of box culvert

Foreshore east of box culvert
NOTES

1 This design profile is typical for this foreshore section and may require appropriate on-site modification.
2 Figure 4-1 needs to be read in conjunction with the text for Brickwharf Road Reserve.
3 Council accepts that crest of this seawall will occasionally be overtopped.
4 Gravel underfooting for load spreading to minimise differential settlement at joints between blocks in the bottom course.
5 Council may wish to consider provision of access for boat owners, by either recessed steps within the seawall or timber stairs built out from the seawall.
5. Davistown Eastern End

5.1 Present Condition

This section of foreshore extends from a T-head jetty at the eastern end of the seabeach-lined foreshore past the enclosed swimming pool to the stormwater outlet. Plate 4 shows photographic perspectives taken from the centre of the subject foreshore looking west and east.

The foreshore exhibits some minor erosion features with a very small erosion escarpment at the back of a sandy shelly beach along the north side of Cockle Channel. The beach is relatively flat at a slope of about 1:10. The erosion escarpment is a maximum of 200mm in height and is backed by relatively flat grassed foreshore reserve. This reserve has a toilet block, picnic facilities and children's play equipment - all of which denote the predominant use of this foreshore area.

At the western end of this foreshore section is a T-head timber jetty. The mooring face of this public jetty is at the edge of the navigation channel. The landward connection of the jetty is flanked by sandstone block wall in need of repair. Pedestrian access to, from and across the foreshore at the base of the jetty has stressed the vegetation and the foreshore edges, and the reserve terrace has eroded from behind the sandstone blocks.

The foreshore immediately to the east of this jetty cuts back, and behind the beach in this location is a drainage depression perpendicular to the beach, which is contributing to the localised erosion in this area.

The foreshore area has several moorings and a private jetty to the east of the T-head jetty at a distance of about 50 metres. About 50 metres further to the east is a dilapidated timber slatted enclosed swimming compound. Evidence of the extent of receding foreshore is exhibited by the location of the landward end of the timber enclosure, which is below high water mark. Examination of historical aerial photography suggests this pool has been subject to periodic sand clearance to maintain adequate water depths. The removal of this sand from the pool would have impacted on the sediment budget in the vicinity.

The eastern extent of this site is defined by a Telecom pit for a cable crossing of Cockle Channel. A stormwater pipe which discharges onto the tidal flat is located immediately to the west of the Telecom pit. The stormwater pipe has recently been extended and a scour hole is developing at the outlet indicating that some formalised treatment in this area will be required.

There is little significant fetch at this site for the generation of wind waves, so the primary driving mechanisms for littoral transport would be tidal currents (not examined in this investigation) and waves generated by boat wakes which would be expected to be random.
5.2 Outline of Options

The options for foreshore treatment of this site include

(i) Continuation of Seabees (Comment while this option was functionally successful for the main foreshore area for Davistown, its continuation is not favoured by GCC for aesthetics and user reasons for this small area)

(ii) Small keyed sandstone block wall (Comment the existing minor erosion exhibited on this foreshore would be exacerbated and probably result in an accelerated loss of the sandy beach in front of such a proposed block wall)

(iii) Mini-groyne field, sand nourishment and access definition (Comment while this option was considered, it was thought that the groyne component would be problematic with potential adverse impacts on the navigation channel, jetty structures and the swimming pool)

(iv) Soft option of access definition, planting, drainage and nourishment (Comment this option is recommended and best addresses the problems for this site. It is discussed in detail in the following sub-section 5.3)

5.3 Recommended foreshore treatment

An overall description of the recommended foreshore treatment of this site generally falls into the category of "soft" protection works. These comprise access definition, drainage improvement, limited nourishment and vegetation plantings. This option best suits the primary users of this section of foreshore being families with small children.

Such "soft" protection works do require monitoring as to performance of the design elements. Some maintenance will be required from time to time dependent on natural causes, vandalism or inappropriate use. It is suggested that for this site GCC produce a public education leaflet for residents and site users, to explain the rationale for the foreshore treatment.

The following text describes the recommended foreshore treatment commencing at the T-head jetty (at the eastern end) and moving along the foreshore in an easterly direction:

- **Near the T-head public jetty**
  Access to and from this jetty and across its foreshore base has stressed the grassed terrace in this area. The sandstone block wall, which armours this mm-headland, has been outflanked, leading to scouring from behind and partial collapse. Access to and from the jetty should be better defined and managed, the small sandstone block seawall reinstated with geotextile fabric behind the wall with graded backfill. A length of the seawall (say the last 3 metres) should have a stepped face to define and permit easy access from the beach to the terraced surface. The seawall should terminate by a tapered return into the terrace reserve.

- **Existing drainage depression**
  To the east of the jetty in the grassed foreshore reserve there exists a drainage depression which would be inundated during extreme elevated water level conditions. This feature would exacerbate beach erosion in this localised area.
It is recommended that this area be filled (or a drainage swale created parallel to the beach) and regrassed to a level in keeping with the overall reserve area. An on-site decision should be made regarding drainage, which should be redirected westward of the T-head jetty.

- **Overall beach and terrace intersect**
  It is recommended that suitably clean and compatible sand be imported to provide limited nourishment for this beach from the T-head jetty to a point east of the pier. It would be appropriate to form a small dune along the edge of the grassed terrace, say 200mm above the level of the grassed terrace.

  It is recommended that this small dune and the grassed terrace edge be planted with appropriate colonising grasses and reeds (and in this respect Soil Conservation Services' advice is commended). This treatment should extend to past the toilet block. Appropriate gaps through this edge vegetation is recommended at suitable locations, for example, opposite the picnic shelter, play equipment and swimming pool. It may also be appropriate to define these access points and the rear of the vegetation with kerb logging.

- **Swimming pool enclosure**
  It is understood that this enclosure is to be retained and possibly repaired. An examination of aerial photography showed that this enclosure has been dragged, extracting sufficient sand to improve water depth for swimming. Sand extracted has probably been removed from the site or deposited to the east of the pool. Such action has created a sediment sink aggravating localised erosion. If such sand extraction is carried out again it is recommended that this sand be distributed along the beach west of the pool, and hence be maintained within the same sediment budget system. This will minimise additional erosion potential of the foreshore.

- **Stormwater pipe outlet east of toilets**
  The proposed treatment at this location is to construct a fitted random rock headwall for the stormwater pipe with rip-rap scour protection extending about 1 metre out from the end of the pipe. The top of the headwall should be at the level of the top of the Telecom pit and the toe of the wall should be at a level at least 0.5 metres below the existing pipe invert. The rocks in the wall should be mortared together to limit the risk of vandalism and the wall alignment should be returned to meet the reeds at highwater mark so as to prevent the wall being outflanked leading to scour behind it.

  Beyond the eastern side of the swimming compound the reed treatment should be continued to the eastern end of the site which is defined by a stormwater drainage pipe running down from Pine Avenue and a large Telecom pit associated with a cable crossing of Cockle Channel.
Typical foreshore looking west towards Davistown Jetty

Typical foreshore looking east towards enclosed swimming pool
6. Empire Bay - North West of Boatshed

6.1 Present Condition

The Empire Bay reserve is on the southern foreshore of Cockle Channel. The reserve is adjoined at its north western boundary by Lot 112 in DP565188. Between this boundary and the boatshed marina operation on Lot 486 in DP727270 the reserve contains a concrete toilet block, a variety of play equipment, a picnic shelter, an electric light pole and several mature trees on a grassed terrace which is generally at a level of 100m AHD. Plate 5 depicts this foreshore area.

At the north western end where there would seem to be limited pedestrian traffic, reeds survive on the high tide line. A beach slopes some 20m out into the channel at a grade of about 1 in 20, beyond this the bed slope steepens as it drops into the main channel.

Further south east along the foreshore the reeds peter out and a foreshore escarpment about 0.5m high becomes apparent. In front of the trees attempts have been made to protect this escarpment by timber plank retaining walls which are falling into disrepair allowing the tree roots to become exposed.

South east of the plank walls is a section of unprotected foreshore to the boundary of the boatshed area, where the beach slope is about 1 in 10 and 15m wide out to the drop off into the main channel.

Within the area of the boatshed allotment the foreshore is protected by a log retaining wall and the beach rapidly steepens and narrows as the channel alignment approaches the shoreline.

6.2 Outline of Options

The options for foreshore treatment of this site include:

(i) The use of Seabees (Comment: this treatment has been functionally successful for much of the northern foreshore of Cockle Channel in the vicinity of Davistown, however its use in this small area is not favoured by GCC due to a combination of aesthetic and also recreational use and access considerations).

(ii) Small keyed sandstone block wall (Comment: the apparent stability of the beach in front of the existing timber walls indicates the suitability of this treatment to form armoured headlands to both protect the root zones of the trees and to define the extent of a small crescent beach to be formed at this site).

(iii) Soft option of access definition, planting and minor land forming (Comment: this option involves the use of reed plantings along the high tide line for bank stabilisation).
the provision of appropriate foreshore access structures at identified access points and minor landforming as required to define a small crescent beach at this site)

6.3 Recommended Foreshore Treatment

The recommended treatment at this site is in essence a combination of options (ii) and (iii). The proposed treatments are aimed at simultaneously defining functional areas within the reserve, providing adequate access to the water and foreshore from the reserve and protecting from present and future threat those trees which GCC wishes to retain in the reserve. The proximity of the boatshed marina to the foreshore means that both the marina operator and the owner of the property will have to be consulted regarding proposed works in its vicinity.

The boundary between the reserve and Lot 486 in DP727270 (on which the boatshed is situated) would appear to be ill defined since no boundary pegs have been found. It is presumed that the existing log wall in this vicinity runs along the boundary, however this has not been confirmed by survey. A survey will be required to clearly define this boundary prior to any construction work on this site.

Figure 6-1 is a sketch only diagram of the plan location and alignments of the proposed foreshore treatments for this site. It should be read in conjunction with the following text which outlines the proposed works commencing at the boatshed (or south eastern end) and travelling generally northwestward along the foreshore.

The existing timber log retaining wall west of the boatshed should be removed and replaced with a keyed sandstone block wall.

Figure 6-2 shows a typical cross-section of the proposed wall for this part of the Empire Bay site. This diagram must be read in conjunction with Figure 6-1 and the following text. It should be noted that the design shown is for a typical cross section and may require appropriate on-site modification at some points along the wall to suit localised anomalies. On-site modifications should be directed and supervised by experienced GCC staff.

The crest level of the wall has been set at 110m AHD which is marginally above the general terrace level in this vicinity. The toe of the gravel under-footing has been set at -0.30m AHD or deeper which will require trenching into the existing terrace. At all locations the toe of the sandstone wall must be below the existing surface level of the adjacent beach.

The south eastern end of the seawall must be physically tied into a substantial foreshore feature such as the western end of the slipway at the boatshed. This will obviously require the co-operation and consent of the marina operator and the site owner. The wall should then continue generally northwestward along the existing wall alignment (and/or the boundary as determined on-site subsequent to survey and construction). The rock seawall should continue beyond the existing wall where it will hook west around the first major tree and be returned at least four metres into the reserve terrace. The alignment of this seawall should be chosen to permit ease of construction while minimising disruption in the root zone of the tree. In this way the seawall will form an armoured mini headland to protect the root zone of the tree while allowing a pocket beach to form to its north west.

A second similar keyed sandstone block wall constructed inside the drip line of the two trees further north west in the reserve would form a second armoured mini headland. The area between these two mini headlands will naturally form a small crescent beach suitable for both...
an access point and a children's play area. Limited land forming and nourishment with a suitably clean and compatible sand would accelerate the formation of the pocket beach, and is therefore recommended.

The second armoured mini headland should be constructed to protect the two large trees in this vicinity, while generally conforming to Figures 6-1 and 6-2. Any on-site modifications required should be directed and supervised by experienced GCC staff. The wall alignment will need to be determined on site with a back hoe. The chosen alignment must allow excavation to the design depth of at least -0.30m AHD while minimizing disruption of the root zone of the two trees which are to be protected. The depth of the trench excavated for the wall must be such that the bottom of the gravel underfooting of the wall will be at least 800mm below the adjacent beach level in its immediate vicinity. Not withstanding the level of the toe of the seawall, the crest level should be 1.10m AHD along its full length. The seawall should be returned at least 4 metres into the reserve terrace at each end.

In the area to the north west of the second armoured mini headland it is recommended that suitably clean and compatible sand be imported to form a small dune along the grassed terrace say 200mm above the level of the existing terrace. Both the dune and the grassed terrace edge should be planted with appropriate salt resistant colonising grasses and reeds (and in this respect the Soil Conservation Services’ advice is commended). This treatment should extend from the armoured mini headland to the boundary with Lot 112 in DP565188. Appropriate gaps through this edge vegetation may be required at locations such as the play equipment or the public toilet block so as to facilitate public access between the reserve and the waterway.
Foreshore to the west with terrace escarpment wall inside tree drip line

Foreshore just west of the marina
NOTES

1. This design layout is not to scale and is subject to on-site decisions regarding alignment and plan location.

2. Figure 6-1 needs to be read in conjunction with the text for Empire Bay - North West of Boatshed.

3. Council accepts that crest of this seawall will occasionally be overtopped.

4. IMPORTANT: This diagram is a SKETCH ONLY and subject to on-site survey and layout.

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Cockle Channel

Intertidal Beach

Beach Nourishment and Dune Stabilisation

Keyed sandstone block walls

Pocket Geotextile area

Trees

Concrete WC

Concrete roof

Possible access point

Onsite decision in consultation with Manna operator and landowner regarding termination of seawall near slipway.
NOTES

1 This design profile is typical for this foreshore section and may require appropriate on-site modification.
2 Figure 6-2 needs to be read in conjunction with the text for Empire Bay - North West.
3 As indicated in Coffey Partners Report No G0625/1-AB.
   The footing should be 1.0m wide with embedment of at least 0.5m.
4 Council accepts that crest of this seawall will occasionally be overtopped.
5 Gravel underfooting for load spreading to minimise differential settlement at joints between blocks in the bottom course.
7. Empire Bay, East of Boatshed

7.1 Present Condition

The Empire Bay Foreshore Reserve forms part of the southern foreshore of Cockle Channel. The section of reserve to the northwest of the boatshed marina operation has been dealt with in the preceding Section 6. This section deals with the remainder of the reserve foreshore, east of the boatshed/marina which is shown in Plate 6. In this area, the navigation channel of Cockle Channel would appear to be migrating to the south, with a consequent impact on the shoreline and options to protect same.

From the boatshed the foreshore follows a curvilinear alignment, running generally south east for about 100m. At this point there is a 1.9m x 0.5m reinforced concrete box culvert discharging into the channel. The culvert outlet features reinforced concrete wingwalls, but no apron nor cutoff wall. The intervening reserve is a near flat grassed terrace at a general elevation of about 1m AHD to 1.10m AHD. There are a few immature gum trees and several power poles scattered around this terrace which is otherwise largely featureless.

The water’s edge of the terrace was formerly defined by a timber plank retaining wall. This wall has now fallen into disrepair, and largely collapsed. Erosion has occurred behind the remnants of the wall, creating an escarpment of irregular height and alignment. There is a narrow sandy beach profile, generally extending less than 10 metres out from the toe of the escarpment. The beach slopes at about 1:7 before it drops off into Cockle Channel. Numerous small craft moor along the foreshore in this vicinity.

Immediately to the east of the box culvert a concrete boat ramp extends out from the foreshore into Cockle Channel. The terrace area behind the boat ramp is occupied by a sealed car parking area, in the extension of Kendall Road into the foreshore reserve. To the east of the boat ramp, a timber public wharf projects out into the channel. The landward end of the wharf is attached to a rock and concrete armoured headland structure.

Beyond the boat ramp and wharf the reserve continues along its curvilinear alignment. It extends some 300m generally eastward to its boundary with an unidentified private property. The reserve here is a nearly flat, grassed terrace, with a general elevation of about 1.00m AHD. There are some power poles along the southern boundary of the reserve which adjoins various houses.

Along the northern edge of the reserve there is an avenue of trees. The trees are of several different species with casuarinas and gum trees predominating. The trees at the eastern end are reasonably mature while those further west are still only young. The majority of the trees are within 2m of the foreshore escarpment which lies to their north. The escarpment is of variable height and alignment.
There is no evidence of any uniform foreshore treatment having been carried out in this section of the reserve. Some parts of the foreshore are unprotected while others have been subjected to various ad hoc treatments with varying degrees of success. The existing ad hoc foreshore treatments would appear to have been carried out by adjacent landowners since their length and location tend to correspond to the projection of the property frontages to the reserve. Since the foreshore treatments are not continuous most have been outflanked leading to erosion behind them and consequent failure.

At the toe of the foreshore escarpment, in the intertidal zone, there is a narrow sandy beach. The beach is of the order of 7 metres wide, sloping out at 1.6 before dropping off into the main Cockle Channel. Numerous small craft are moored along the entire length of the foreshore.

In general the reserve appears to be used primarily by local residents for access to the moorings along the foreshore, to the boat ramp, to the public wharf and to the boathed marina operation. The erosion experienced to date does not appear to have seriously diminished the amenity of the reserve for its primary users. It has however, led to some minor reduction in the plan area of the reserve terrace and has placed some of the trees under threat. If allowed to continue unabated foreshore erosion would be expected to further threaten these assets.

Cockle Channel is not a channel with a stable natural alignment. It would appear to have been dredged for improved navigation on at least one previous occasion. A detailed study of the channel, including its history and its rate of realignment, was beyond the scope of the current brief. For this reason no definitive statements can be made about the rate at which the channel is likely to move further south and threaten the existing foreshore in this vicinity. It is felt however, that since this part of the foreshore forms the outside of a bend that the tendency of the channel would be to migrate further south unless constrained by major training works.

In consultation with the the NSW Public Works, Coast and Estuaries Branch (C&E) an analysis of this section of Cockle Channel was carried out. This assessment involved an examination of C&E's recently completed hydrographic survey of the existing channel, and an analysis of foreshore escarpment locations from 1941 to 1985, using photogrammetric techniques. The results of this assessment have been reported to MHL by C&E in the following terms:

"It appears that the maximum erosion is around 4m - 5m from 1941 to 1971 with little movement from 1971 to 1985."

7.2 Outline of Options

There is a range of options available for foreshore protection works at this site. One approach would be to construct major training works to simultaneously protect against both foreshore erosion and channel migration. To design such a structure, a detailed study of the channel and the morphodynamic processes shaping it would be required, so as to ensure an adequate basis for design. Such a structure would be likely to be large, expensive, and significantly impact on the aesthetics and mooring amenity of the foreshore.

A second, more favoured, approach would be to construct a wall of sufficient height to protect the foreshore escarpment from further erosion. This option is more favoured given the relatively low rate of foreshore erosion detected by the photogrammetry. It should be noted
however that should any further southward migration of the channel occur, it would place this wall under threat by undercutting its foundations. For this reason it may be necessary to take steps to realign the channel further to the north. By moving the channel further north the width of the beach at the toe of the wall could be increased. Increasing the beach width would be expected to buy time before channel migration southward placed the wall under threat. It would be essential with this option to monitor the rate of any channel migration, by regularly surveying fixed cross-sections. This should allow sufficient time to undertake the necessary channel re-alignment, so as to prevent any threat to the wall arising.

7.3 Recommended Foreshore Treatment

The recommended strategy for the Empire Bay-East site is to integrate management of the navigation channel with the foreshore protection works. It is recommended that consideration be given to dredging an improved navigation channel in this reach along an alignment generally to the north of the existing channel. The spoil from these dredging works would be used to fill in the existing channel alignment and to top up the beach profile in front of the existing foreshore alignment.

The advice provided by C&E on the history of foreshore erosion in this vicinity, suggests that a seawall built along the existing foreshore alignment may not be under immediate threat of collapse due to channel migration. For this reason, it would be acceptable to construct a wall along the existing foreshore alignment, without first realigning the channel northward provided regular, accurate monitoring of the channel's location is undertaken. The monitoring should be undertaken prior to wall construction to establish baseline conditions. A series of suitable channel cross-sections should be selected in consultation with C&E for use as monitoring lines. The selected cross-sections should be accurately located and permanently marked on the ground at the site. Regular surveys must then be carried out along exactly the same alignments. Surveys should initially be carried out at monthly intervals. Based on the results of the initial surveys the monitoring interval may later be adjusted. In the event of fluctuations in channel location being disclosed by surveys, expert advice should be sought immediately, to assist in interpreting the significance and implications of the fluctuations.

Figure 7-1 shows a typical cross-section of the proposed wall for this part of the Empire Bay site. The drawing must be read in conjunction with this text. It should be noted that the design shown is for a typical cross-section and may require appropriate on-site modifications at some points along the wall to suit localised conditions. On-site modifications should be directed and supervised by experienced GCC staff.

The crest level has been set at 110m AHD which is marginally above the general terrace level in this vicinity. The toe of the sandstone block wall has been set at -0.50m AHD or deeper. The bottom of the gravel underfooting will therefore be at -0.80m or deeper. The toe of the wall and the gravel underfooting will have to be trenched into the sandy beach. The trench must be excavated at least 0.8m deep so that the toe of the sandstone wall is always at least 0.5m below the existing beach level in its vicinity.

The alignment of the wall will have to be selected on site to suit the conditions as they exist. The alignment should follow the general shape of the existing foreshore, approximating it with a series of smooth curves. In choosing the alignment, consideration must be given to tying the wall to substantial foreshore features, such as the culvert wingwall and the rock and concrete armoured headland at the base of the wharf. The alignment must also allow...
installation of the free draining granular material behind the wall while minimising disruption to the root zone of the trees along the foreshore

GCC accepts that during periods of elevated water levels the wall may be periodically overtopped. Overtopping could lead to some scour and damage to the grassed area behind the wall. Periodic maintenance of this area, including topsoiling and turfing will be necessary. The overall drainage from the reserve will need to be assessed and accommodated by GCC in association with its works.

It is anticipated that there will be some loss of sand from the beach profile in front of the (near) vertical wall, due to the higher reflected wave energy. For this reason the beach profile in front of the wall should be topped up. Material from the excavation of the footings and from the dredging operations should prove suitable.

It is important to note that construction of the rock seawall alone will not secure protection of this foreshore if the navigation channel continues to migrate to the south. Channel re-alignment via dredging must be carried out either in tandem with the rock seawall protection works, or as soon as channel monitoring discloses any net southward migration of the channel alignment.
Eroded terrace west of the ramp and culvert

Eroded terrace east of the boat ramp
Seawall to comprise of large rough cut sandstone blocks (0.5 tonne to 3 tonne) keyed in random pattern and dry grouted

Seawall crest at 1.10 AHD

Larger blocks used for bottom row of seawall with weep holes provided by spacing blocks with 20 to 50mm gaps

Gravel underfooting at -0.80 AHD or deeper

Gravel underfooting for load spreading to minimise differential settlement at joints between blocks in the bottom course

Notes

1. This design profile is typical for this foreshore section and may require appropriate on-site modification.

2. Figure 7-1 needs to be read in conjunction with the text for Empire Bay - North East.

3. As indicated in Coffey Partners Report No GD6251-AB.
   The footing should be 1.0m wide with embedment of at least 0.5m.

4. Council accepts that crest of this seawall will occasionally be overtopped.

5. Gravel underfooting for load spreading to minimise differential settlement at joints between blocks in the bottom course.
8. Gosford Waterfront

8.1 Present Condition

This foreshore section of Gosford's waterfront is that part of the shoreline east of the Broad Water in the vicinity of the boat launching ramp and the Olympic Swimming Pool. The shoreline under consideration (depicted in Plates 7 and 8) extends from an existing sandstone block seawall (approximately 40 metres north of the boat ramp) at the northern end, past the swimming pool to the embayment used by the Gosford Sailing Club at the southern end (approximately 250 metres south of the boat ramp).

Between the boat launching ramp and the sandstone block seawall to the north, the existing rubble sandstone rock protection has degraded. A rip-rap berm in the tidal zone slopes into the waterway at approximately 1:10. Larger sandstone rocks line the foreshore escarpment which has cut back a distance of some 4 metres beyond the (extended) line of the existing northern seawall. The erosion escarpment is topped by a grassed terrace at a level of 1.3m AHD. Immediately north of the 2-3 lane boat launching ramp is a timber T-head jetty.

To the south of the boat ramp the degraded rock protection (similar to that described above for the northern shoreline) continues until one reaches the brick wall compound of the swimming pool. There are a number of mature trees along the edge of the erosion escarpment which are under stress and in danger of being lost.

On the lake side of the swimming pool's brick building a 45m length of seawall comprising concrete cast blocks lines the shoreline with a rip-rap berm beaching out from the toe. In conjunction with this concrete-block seawall there are numerous drainage outlets from the swimming pool complex.

The grassed terrace west of the swimming pool compound varies in width as the western boundary of the swimming pool compound juts in and out. Pedestrian traffic along the top of the foreshore section has worn the grassed surface in some locations. It is reported that this terrace is also subjected to some wave inundation during extreme events.

From the southern end of the swimming pool to the Sailing Club embayment the foreshore protection essentially comprises a steep seawall of large sandstone rock placed in position. The terrace (at the level of approximately 1.3m AHD) behind this seawall is grassed although stressed from pedestrian traffic, and inundation from both elevated lake water and local runoff particularly from the swimming pool roof. This seawall also has a rip-rap toe berm beaching out below the tidal zone.
8.2 Outline of Options

While there is a range of options for foreshore protection at this site GCC has indicated its preferred design option GCC's preferred option for the section north of the boat ramp is a cut sandstone wall; while south of the boat ramp is for a mortared keyed sandstone wall using rough shaped boulders.

8.3 Recommended Foreshore Treatment

- **North of the Boat Ramp**
  Figure 8-1 shows a typical cross-section of the recommended foreshore treatment for the area between the boat ramp and the existing square cut sandstone block wall to its north  Coffey conducted a stability analysis for the Figure 8-1 wall  Based on their analysis they produced a design chart which they designated drawing No G06251/1-6  This design chart and its covering letter are attached immediately following Figure 8-1  All three must be read in conjunction with this text. It should be noted that this design is for a typical cross-section and may require appropriate on-site modification at some points along the length of the wall  On-site modifications should be directed and supervised by experienced GCC staff  The recommended treatment in this section of foreshore is a near vertical masonry wall  The wall should be constructed using a combination of the concrete masonry units currently installed in front of the swimming pool complex and suitably sized square cut sandstone blocks cut by the GCC stonemasons.

  The footings of the new section wall should be built using sandstone blocks cut to have the same bearing surface area and with the same depth of embedment as those in the existing square cut sandstone block wall  Courses of the concrete masonry units should be used to build the wall up to a level of 1.5m AHD which corresponds to the crest level of the existing wall  The face of this seawall should also correspond to the same slight (off-vertical) slope of the existing wall.

  It may be necessary to cut additional sandstone blocks to suit, and either to be interspersed amongst the concrete units to make up the numbers, or to be placed on the top of the wall as a capping course to make up the height of the wall to the design level  Some provision will need to be made for drainage from the back of the wall either by leaving 20mm to 50mm gaps between adjacent blocks or by providing weep holes in the wall  Experienced GCC staff supervising wall construction should decide on-site the most appropriate construction details as circumstances dictate.

- **South of the Boat Ramp**
  Figure 8-2 shows a typical cross-section of the recommended foreshore treatment for the Gosford foreshore between the boat ramp and the Sailing Club embayment  Coffey conducted a stability analysis for the Figure 8-2 wall  Based on their analysis they produced a design chart which they designated drawing No G06251/1-5  This design chart and its covering letter are attached immediately following Figure 8-2  All three must be read in conjunction with the text. It should be noted that this design is for a typical cross-section and may require appropriate on-site modification at different points along the length of the wall  On-site modifications should be directed and supervised by experienced GCC staff.

  The recommended treatment at this site is a keyed sandstone block seawall, comprising large rough-cut sandstone blocks varying in size from say 0.5 tonne to 4.0 tonnes  The blocks are to be keyed into a random pattern and mortared as required to prevent removal  The larger
blocks should be placed at the base with weep holes provided by spacing blocks with a 20mm to 50mm gap. In respect to the provision of weep holes it is imperative that mortar not be used to plug the gaps intended to act as weep holes in the lower courses. It is proposed to reshape as required the existing sandstone rocks already on-site and to import others as required.

- **Overall**
  To allow construction of the new walls along the chosen alignment, which generally conforms to the existing shoreline, the rock and concrete units currently along this alignment must first be removed and stockpiled in adjacent areas. The details of the wall alignments and returns in the vicinity of the boat ramp will need to be determined on-site. Considerations of utility, public access and construction access will have to be carefully balanced. Access to and from the T-Head jetty immediately north of the ramp must be maintained which may necessitate partial reconstruction of the jetty to suit whatever wall alignment is chosen.

  The rip-rap at the toe of the existing escarpment can be pushed out onto the intertidal area in front of the wall from where it can later be dragged back to armour the toe of the new walls. Additional rip-rap will be required in the vicinity of the boat ramp. It is proposed to shape the rip-rap on the southern side of the boat ramp to create a cobbled beach profile in this area. The creation of a beach profile south of the boat ramp will shelter the boat ramp by promoting breaking of waves further south, rather than directly on to the ramp. Wave energy affecting the ramp and its operation should be monitored during storm events and further safety measures taken if deemed necessary.

  The larger concrete and sandstone units which will be re-used in the wall construction should be stockpiled on the terrace areas where cutting and shaping operations may be conveniently carried out. The stockpiling of masonry on the terrace areas should not obstruct access for the machinery used to excavate the trench for the footings, to shape the face of the embankment prior to the placement of the geotextile, to lift the boulders for placement in the wall, for backfilling behind the wall and for reshaping of rip-rap armouring the toe of the wall.

  It is understood that GCC is to rationalise the alignment of the western or rear boundary of the pool complex. This rationalisation should be carried out prior to other works so as to maximise the space available for construction activities. The realignment of this boundary and the selection of an appropriate wall alignment in this vicinity will be expected to create an adequate corridor to accommodate the proposed cycleway in this area.

  In relation to making provision for drainage pipes passing through the wall to discharge to a point at least 200mm beyond the face of the seawall it should be noted that the proposed wall will be a semi-rigid structure. Some differential settlement of the footings would be expected which would lead to movement and re-alignment of the rest of the wall including the areas surrounding pipes passing through the wall. The use of flexible pipes or flexible joints in the pipeline to accommodate wall movement would be prudent. It was noted during the field inspections that some drainage pipes currently discharge behind the rear face of the existing foreshore protection armour units. It is stressed that this practice should be abandoned in future and all drainage pipelines should pass through the wall to discharge in front of it as outlined above.
Foreshore north of boat ramp. New wall to be married into existing wall (centre of photo)

Foreshore to the south with boat ramp mid photo
Typical foreshore looking north

Existing foreshore protection looking north
NOTES

1. This design profile is typical for this foreshore section and may require appropriate on-site modification.
2. Figure 8-1 needs to be read in conjunction with the text for Gosford Waterfront North.
3. For reasons of safety/self reliance footholes recessed into the sandstone seawall face should be provided at convenient intervals.
4. IMPORTANT - Design is subject to analysis and review undertaken by Coffey Partners (Gosford) re retaining structural stability and concurrent reference should be made to Coffey's letter dated 11 March 1993 and Coffey's Drawing No G0625/1-6 (copy attached).
At the request of Mr John Murtagh of Manly Hydraulics Laboratory, Coffey Partners International Pty Ltd have carried out a stability assessment of rock wall stability for a vertical section of sea wall at the Gosford foreshore. Stability analyses were undertaken for walls of varying heights. The results of these calculations are graphically presented on Drawing N° GO625/1-6 attached.

It has been assumed in the preparation of the attached chart that the water table may rise to the full height of the wall. Other assumptions made in the preparation of the design chart are given in the notes presented on the drawing.

The rock wall design chart in Drawing N° GO625/1-6 has been prepared for the design of vertical face walls. It is recommended that this chart be used for walls with face slopes of vertical to 1H 2V. For walls with face slopes less than 1H 2V, use Drawing N° GO625/1-5.

Should you have any questions regarding the above matter, please do not hesitate to contact the undersigned.

For and on behalf of
COFFEY PARTNERS INTERNATIONAL PTY LTD

[Signature]

B A STEPHENS

Encl Important Information About Your Geotechnical Report
Drawing N° GO625/1-6
1. Backfill is to be granular, free draining and compacted. Weepholes are to be provided by spacing blocks with 20-50mm gaps.

2. Foundation to be approved for an allowable bearing pressure of 200kPa prior to construction.

3. Where surface slope of retained material is between 10:1 and 4:1 the wall base dimension is to be increased by 0.5m.

4. Maximum water table height is assumed to be the top of rock wall.

5. Rock is to be of sound durable sandstone or other approved material with a minimum dimension of 0.5m.

6. Rocks shall be placed in such a manner that they are stable and interlocking and laid roughly coursed and bedded on the broadest base.
NOTES

1. This design profile is typical for this foreshore section and may require appropriate on-site modification.

2. Figure 8.2 needs to be read in conjunction with the text for Gosford Waterfront.

3. There are numerous drainage outlets appearing in the existing degraded/rubble seawall. On-site provision should be made to accommodate all drainage from their source to outlets 200mm beyond the face of the seawall.

4. For reasons of safety/self reliance, footholes recessed into the sandstone seawall face should be provided at convenient intervals.

5. IMPORTANT - Design is subject to analysis and review undertaken by Coffey Partners (Gosford) re retaining structural stability and concurrent reference should be made to Coffey's letter dated 2 February 1993 and Coffey's Drawing No G0625/1-5 (copy attached).
ATTENTION MP CHRIS ARGAET

Dear Sir:

RE: PROPOSED SEAWALL, SITE 3A GOSFORD WATERFRONT - ROCKWALL DESIGN

At the request of Mr. Will Strachan of Manly Hydraulics Laboratory, Coffey Partners International Pty Ltd have carried out an assessment of rockwall stability for the proposed seawall at Gosford foreshore. Stability analyses were undertaken for rockwalls of various heights. The results of these stability analyses are graphically presented on Drawing No. G0625/1-5 attached.

It has been assumed in the preparation of the attached design guide that the water table will rise to the full height of the wall.

Other assumptions made in the preparation of the design chart are given on the notes to the drawing.

Should you have any questions regarding this matter please do not hesitate to contact the undersigned.

For and on behalf of
COFFEY PARTNERS INTERNATIONAL PTY LTD

D A STEPHENS

Encl. Important Information About Your Geotechnical Engineering Report Drawing No. G0625/1-5
Notes

1. Backfill is to granular, free draining and compacted. Weepholes are to be provided by spacing blocks with 20-50mm gaps.

2. Foundation to be approved for a safe bearing capacity of 200kPa prior to construction.

3. Whole surface slope of retained material is between 10:1 and 4:1 the wall base dimension is to be increased by 0.5m

4. Maximum water table height is assumed to be the top of rock wall

5. Rock is to be of sound durable sandstone or other approved material and at least 0.5m in plan area.

6. Rocks shall be placed in such a manner that they are stable and interlocking and laid roughly coursed and bedded on the broadest base.
9. **Saratoga Oval Foreshore**

9.1 **Present Condition**

Saratoga lies on a promontory which forms both the southern foreshore of the main basin of Brisbane Water as well the eastern foreshore of Paddy’s Channel (this channel links Brisbane Water to Broken Bay). The Saratoga oval reserve occupies the north western edge of the promontory. The existing foreshore is depicted in Plate 9.

At the western end of the reserve, in the vicinity of the club house of the local sailing club, the foreshore features dense vegetation. A stand of casuarinas occupies the dry land above the high tide line and extensive stands of mangroves cover the intertidal flats extending both north and west from this part of the reserve. The sailing club house is nestled amongst this vegetation which is providing excellent foreshore protection with no foreshore erosion evident in this area.

To the east of the reserve area described above is a large open grassed terrace. The terrace area accommodates a cricket pitch, playing fields, children’s play equipment, park benches and a sand/soil boat ramp access adjacent to the sailing club. This area is used for such land based recreational activities as organised sporting fixtures, children’s use of the play equipment, public perambulation and sightseeing. The terrace is also used as a rigging and marshalling area for the sailing club on race days. The terrace has a general elevation of 1.0m AHD.

Along the foreshore of this area there are extensive but intermittent clumps of reeds growing along the edge of the grassed terrace. An erosion escarpment of variable heights less than 0.5m runs from the boat ramp to a rocky headland some 200m to the east. The erosion is currently undercutting many of the reeds along the terrace edge which has a general elevation of 0.7m AHD.

Below the escarpment is a clayey sand intertidal flat extending northward at a slope of 1:120 which forms a wide beach at low tide. At the western end the intertidal flat extends out over 100m before dropping off into deeper water. The width of the intertidal flat gradually diminishes towards the eastern end where rock extends out into the water. The remnants of a timber jetty extend across part of the intertidal flat from the grassed terrace area out to the deeper water beyond the drop off. The decking of the jetty has been removed leaving only the two lines of piles in place.

To the east of the level playing field area of the grassed terrace the reserve narrows somewhat to a strip of land sloping down to the foreshore escarpment from the boundaries of the adjoining residential properties further up the hill. A number of mature trees (including two large Port Jackson Figs) are scattered throughout this area of the reserve. The clayey sand intertidal flat discussed earlier is interrupted in this part of the reserve by a sandstone outcropping from the hill which then dominates the foreshore as it progresses further east.
Close to the western end of the sandstone outcropping a large fig tree is dangerously close to the erosion escarpment and has its roots exposed. The tree is exhibiting symptoms of stress and is in danger of collapse if erosion is allowed to further undermine it. The protection of this tree represents the eastern boundary of the proposed works.

The section of the Saratoga oval reserve foreshore which requires protection has a northern exposure with a fetch length for wave generation of 4600m. For a 5 year annual recurrence interval at this site the design still water level has been assessed as being 1 06m AHD and the design wave as 0.64m.

Subsurface investigations at this site carried out by Coffey involved excavating, logging and sampling of three hand auger holes at approximately 50m intervals, with six dynamic cone penterometer tests carried out at approximately 25m intervals, in the foreshore area between the boat ramp and the sandstone outcropping. Their report describes the existing subsurface conditions at Saratoga in the following terms:

Based on the results of the hand auger holes the subsurface profile at the Saratoga site may be summarised as follows:

- **TOPSOIL**
  - (SP) Silty Sand, fine grained, brown, some roots, observed
  - unit thickness varied from 0 to 0.2m, overlying

- **ALLUVIUM**
  - (SP) Sand, fine grained, brown and orange-brown, some silt, some shells, observed
  - unit thickness varied from 0.7m to 1.6m, overlying

- **ALLUVIUM/RESIDUAL**
  - (SC) Clayey Sand, fine grained, light grey, moist, dense,
  - unit was observed between 0.7m to greater than 1.8m depth

They further report that the assessed allowable bearing pressure for a wall at Saratoga with 1.0m wide footings embedded to 0.5m is 150kPa.

### 9.2 Outline of Options

There are several options available for foreshore protection treatments at this site. In considering the various options a balance needs to be achieved between the technical practicality of treatments and their impact on the amenity of the reserve for its users. The options considered for this site are:

1. **Soft option of mangrove establishment** (Comment: the establishment of mangroves on the intertidal flats would in time significantly reduce wave attack on the foreshore, promote accretion of sediments and in the long term stabilise the reserve area above the existing intertidal area. It would be a long term solution requiring in the short term, protection of the mangrove seedlings while they become established (Reference: text on artificial mangrove plantings - see Appendix B). This option will, however, inhibit the sailing club's current free access to the water over the entire waterfront and also cut off the panoramic northern views currently available from most areas of the reserve.)

2. **Establishment of a cobble beach** (Comment: cobbles are gravel particles ranging in size from 60mm to 250mm which are much less prone to transport than sand sized material.)
Nourishment of the area in front of the existing erosion escarpment with gravel of this size would be expected to yield a stable beach profile over which pedestrian access for either beach combing or boat carrying would be quite practicable. Cobble beaches are not common in Australia where sandy beaches are the norm and aesthetic objections from reserve users could be expected. The volume of cobbles required would be of the order of 600m$^3$).

(iii) Seabees (Comment: This option would be a functionally viable treatment for the Saratoga foreshore, however it is not favoured by GCC for aesthetic and user reasons)

(iv) Mini-groyne field and sand nourishment (Comment: while this option should be considered, the groyne component is problematic with potentially adverse impacts on other downdrift areas. This option would require a detailed assessment of the processes if it were to be further considered)

(v) Keyed sandstone block wall (Comment: this wall could be sloped, stepped or near vertical. A sloped or stepped wall would facilitate pedestrian access including boat carrying from the terrace to the intertidal area. Provision of a near vertical wall would tend to encourage people to use defined access points such as the existing boat ramp and any additional ramps, steps or jetties which are provided. Provision of a near vertical wall would be expected to maximise scour at the toe of the wall).
Typical foreshore looking west

Typical foreshore looking east
10. Brisbane Water Drive (Southern End of Tascott Straight)

10.1 Present Condition

This section of foreshore is depicted in Plate 10. It is in the area north of Koolewong and south of Tascott. The foreshore area is east of Brisbane Water Drive and at the southern end of a road section known as Tascott Straight. (It does not include the foreshore reserve area in the centre of Tascott Straight behind the mud flats refer to Section 11).

The specific section of foreshore extends from private properties at Koolewong in the south for a distance of some 450 metres to the west and north. The terraced area behind the shoreline has a typical elevation of 1.0m AHD and sandstone rock protection previously carried out is now degraded. The existing sandstone rip-rap comprises rock varying in size from 0.5 to 1.2m. The typical cross-section throughout this area is at a slope of 1:3 extending out to a depth of about 0.0 AHD, below which the bed slope is about 1:50.

The foreshore area behind this rock protection partly comprises a bitumen sealed car parking area primarily to serve the boat launching ramp, dinghy storage area, and other users of the two small public jetties. The lakeside edge of the sealed car park has fretted away under combined wave attack and without any provision for surface water runoff. This in turn has resulted in a small gully between the car park and the landward side of the existing degraded seawall.

The rest of the foreshore behind the existing rock protection in this reserve consists of a grassed bank planted with casuarinas. Overtopping of the degraded rock seawall in the grassed areas has led to erosion of the bank and gullying behind the wall. GCC Engineer, Mike Alsopp advised that this seawall had been installed with an early geotextile fabric filter, which has since broken down.

This foreshore is exposed to north-east to eastern sector with a fetch distance of up to 4000 metres. During episodes of elevated water levels, waves break over the existing seawall, then runup over the sealed car park and/or grassed terrace. This wave action on the unprotected edge of the seal has exacerbated the erosion of the edge of the sealed car park (and grassed embankment).

Design water levels and wind wave heights with a range of recurrence intervals have been assessed for this site and used in the design. The crest level has been set to prevent overtopping by waves for an event with a 5 year recurrence interval with 0.7m waves generated by 62 kph winds blowing over a 4000m fetch from the ESE for 28 minutes. More extreme events may cause overtopping of the proposed revetment and so care has been taken in the design to allow for scour protection and drainage at the rear of the wall to prevent a recurrence of the existing failure mode.
10.2 Outline of Options

While there is a range of options for foreshore treatment of this site the most feasible is an enhancement of the existing rock protection by reforming it into a revetment with a higher crest level and improved car park edge treatment. This recommended foreshore treatment was discussed on site and agreed, in principle, by GCC personnel and PWD engineers from Coast and Estuaries (C&E) Branch.

10.3 Recommended Foreshore Treatment

Figures 10-1, 10-2 and 10-3 show typical cross-sections and detail of the recommended foreshore treatment for this site. These diagrams need to be read in conjunction with this text. It should be noted that these design profiles are typical and may require appropriate on site modification which should be directed and supervised by experienced GCC staff.

The recommended treatment at this site is a rubble revetment structure which has been designed to withstand limited overtopping for events more extreme than the five year event. Available information indicates that the water level can be at the same level of the car park. The crest level of the revetment has been set at 1.7m AHD which is the limit of wave runup for the five year event coinciding with the top of the tide. For more extreme events coinciding with the top of the tide some moderate overtopping would be expected. This is unlikely to be a problem with limited use of the car park at such a time. Two treatments at the back of the wall have been adopted as part of this minimum design structure to deal with the overtopping.

In paved areas the treatment shown at Figures 10-1 and 10-2 should be adopted. The keyed concrete kerb and gutter in conjunction with an asphaltic concrete overlay to the existing spray seal has been designed to control overflow waters and prevent infiltration of this water to the car park pavement. In this manner the fretting of the seal and scouring of pavement material behind the wall will be prevented.

In grassed areas the treatment shown at Figure 10-3 should be adopted. The crushed rip-rap layer underlaid with geotextile fabric is to be continued 1500mm behind the rear face of the revetment. This area is then to be topsoiled and regrassed. This treatment will lend this area the same appearance as the rest of the grassed bank. In the event of minor overtopping the sandstone layer will provide a subsoil drainage path below the grass root zone. In the event of major overtopping stripping the grass the rip-rap will armour the bank and prevent scouring of the bank behind and below the revetment.

In constructing the new revetment the existing rock will need to be cleared from the construction area. Some may be dragged up to the car park but most should be stacked out beyond the existing toe from where it can be dragged back on to the revetment as it is built up. It is likely the existing toe armour units will be difficult to move and where possible they should be left in place and used as the inner layer of armour rocks.

The geotextile fabric has been incorporated into the revetment to prevent migration of fines from the bank and undercutting of the revetment by wave action. It may be necessary to trim the bank profile and place some compacted fill to achieve an appropriate alignment prior to placement of the geotextile which will need to be returned at the toe and crest as shown in the Figures.
The rip-rap layer has been incorporated to protect the geotextile from mechanical damage and should be placed over the geotextile and behind the armour rocks stacked out beyond the existing toe. As the rip-rap is built up and trimmed to the design (1:3) slope the armour rocks can then be dragged back on to the rip-rap in two randomly placed layers which should generally conform with the design (1:3) slope of the structure. In this manner the structure should be built up to a crest level of RL 170m AHD. The revetment should be two rocks wide at the crest with a gap to the kerb alignment.

Typically the armour rocks need to be 0.2 to 0.5 tonne in size, however larger (1 to 2 tonne) rocks may be occasionally and appropriately placed occupying the width of both layers.

The concrete kerb and gutter should then be placed with the geotextile draped into the trench for the key. After a suitable curing period for the kerb and gutter selected rock armour should be placed over the rip-rap between the back of the kerb and the back of the crest. The rocks in this area should be selected or trimmed to fit the gap and be sufficiently large enough to prevent vandalism. They should not be mortared into place to allow maximum porosity for water backflow. An asphaltic concrete overlay should then be placed on the car park from the lip of the gutter section. Adequate compaction of the asphaltic concrete is essential to prevent ingress of water to the pavement leading to failure of the bank behind the revetment as happened to the existing structure.

On-site modification of this typical design profile and edge detail will obviously be necessary at some locations, including the boat launching ramp and jetty sites.
Existing rock protection, note eroded edge of car park

Existing rock protection ineffectual with grassed terrace eroded
Limit of Available Survey

2 rocks wide

Sandstone Armour Rock - typically 0.21 to 0.51 random placed in two layers with a combined thickness of 900mm

Excess existing rock (dotted) beyond toe may be left in place

SCALE 1:50

NOTE
1 This design profile is typical for this foreshore section and may require appropriate on-site modification
2 Figure 10-1 needs to be read in conjunction with text in Section 10 South Tascott
NOTE
1 This design profile is typical for this foreshore section and may require appropriate on-site modification.
2 Figure 10-2 needs to be read in conjunction with text in Section 10 South Tascott.
Limit of Available Survey

1500
2 rocks wide

RL 170

Grassed area

To be top soiled and regressed

Sandstone Armour Rock - typically 0.2t to 0.5t random placed in two layers with a combined thickness of 900mm

Excess existing rock (dotted) beyond toe may be left in place

Graded rip rap layer (typically 400mm thick)

Rip rap layer to extend 1500 behind rear face of wall for over topping protection

Geotextile

Toe of Geotextile returned upwards wrapped over inner toe armour

AHD 0.00

NOTE
1 This design profile is typical for this foreshore section and may require appropriate on-site modification
2 Figure 5A-3 needs to be read in conjunction with text in Section 10 South Tascott

SCALE 1:50

Offshore slope (1:50)
11. Brisbane Water Drive (Tascott Straight)

11.1 Present Condition

This section of foreshore fronts a parkland reserve known as the Koolewong Waterfront Reserve, on the eastern side of Brisbane Water Drive roughly in the centre of the road section known as Tascott Straight. The foreshore is depicted in Plate 11. The reserve is reported to be popular as a picnic site, a driver rest area and tourist lookout, and as a land base for sailboarders and boaters using the adjacent part of Brisbane Water. GCC has made it clear that the maintenance of both the visual amenity and the free access to the water of reserve users at this site are important design criteria for any foreshore protection.

The reserve generally is reported to be composed of a fill layer which was previously bounded by a timber log retaining wall at the water's edge. The fill layer is overlying the alluvium of the pre-existing intertidal flats. The reserve generally slopes toward the water to the east with the general terrace level around 1.05m AHD and the level at the top of the existing foreshore escarpment being about 0.90m AHD.

The 1 year annual return interval (ARI) still water level at this site is 0.99m AHD while the 5 year ARI still water level is 1.06m AHD which indicates that sections of the reserve will be subject to periodic tidal inundation. The 5 year ARI design wave at this site is 0.59m which if generated at high tide conditions would reach the foreshore unbroken whereupon it would plunge on to the grassed terrace causing scouring as its energy was dissipated.

The terrace of the reserve features sealed car parking areas set back and adjacent to the main road, as well as children's play equipment, a picnic shelter, a toilet block and numerous mature and immature trees scattered throughout the grassed area. The foreshore erosion is undercutting the roots of some trees which are therefore in danger of collapse.

The southern end of this site is defined by a sandstone rock outcrop which significantly raises the terrace level in its vicinity to around 1.72m AHD. To the south of the sandstone outcrop is the site discussed previously in Section 10. The northern end of this site is defined by the southern end of an existing sandstone rock revetment type structure which fronts the foreshore. This section of foreshore was not included in the brief for the current study.

A wide intertidal flat with a slope of about 1:100 extends eastward from the reserve into Brisbane Water. At low tide this flat forms an intertidal beach which has no berm or dune system above high tide level. There are a few scattered mature mangroves as well as seedlings on the intertidal flat, which is otherwise free of vegetation. While the absence of screening vegetation on the intertidal flats allows reserve users to enjoy both panoramic views over Brisbane Water as well as free access across the flats to the water, it is also the reason the foreshore is currently exposed to wave attack.

Coffey conducted sub-surface investigations at this site including excavating, logging and sampling of four hand auger holes of varying depths at approximately 50m intervals and
conducting seven dynamic cone penetrometer tests at about 25m intervals. Based on their investigations Coffey concluded the following soil profile -

**TOPSOIL/FILL**

(CL) Sandy Clay, low plasticity, bricks and sandstone observed in matrix, some peaty zones, observed unit thickness varied from 0.3m to 0.6m; overlying

**ALLUVIUM**

(SP) Sand, fine to medium grained, light grey, trace of fines, medium dense to dense, observed unit thickness varied from 0.9m to 1.2m, overlying

**ALLUVIUM/RESIDUAL**

(SC) Clayey Sand, fine to medium grained, light grey, medium dense to dense, unit was observed to depths of 1.7m

Coffey assessed that the allowable bearing capacity at the Koolewong waterfront reserve site is 150kPa

### 11.2 Outline of Options

There is a range of options for foreshore treatment at this site which while technically feasible from a foreshore protection viewpoint do not satisfy GCC's requirement to maintain both views and access from the reserve to Brisbane Water and have therefore been dismissed from further consideration. The dismissed options include regeneration of mangroves screening the foreshore, sand nourishment including formation of a stabilised dune and gravel nourishment to form a cobble beach and dune.

GCC has also directed MHL to develop a design for a sandstone block wall with the crest level at or about the level of the existing terrace so as to limit the depth of any additional fill which could further stress the existing trees. This level would not interrupt views or access from the reserve to the water.

### 11.3 Designed Foreshore Treatment

Figure 11-1 shows a typical cross-section of the designed foreshore treatment for Koolewong waterfront reserve at Tascott. This diagram needs to be read in conjunction with the following text. It should be noted that the design is for a typical cross-section and may require appropriate on-site modification at different points along the length of the wall, as for example, in the vicinity of the existing drainage pipe at about the mid point of the wall. On-site modifications should be directed and supervised by experienced GCC staff.

The recommended treatment at this site is a sandstone block seawall, comprising large rough-cut sandstock blocks varying in size from say 0.5 tonne to 3.0 tonne. The blocks are to be keyed in a random pattern and dry grouted. The larger blocks should be placed at the base with weep holes provided by spacing blocks with a 20 to 50 mm gap. It is proposed to shape some of the sandstone rocks so as to provide the 10m wide footing required.

The GCC directive for this site that the crest level of the structure be limited effectively means adopting a seawall design which will be regularly overtopped. The design has however been developed so as to limit both the incidence of tidal inundation of the reserve and the scour damage behind the structure caused by waves breaking over it. The crest level has been set at
1 m AHD which corresponds to a design still water level with a 10 year ARI. However some overtopping would be expected more frequently due to wave action coinciding with peak water levels of lesser ARIs. The design is based on the wave breaking when it reaches the front face of the wall and hence plunging over the top of the wall. The proposed concrete pathway at the rear face of the wall/terrace is an integral part of the design and is intended to armour this part of the wall to limit scouring of the graded backfill. The pathway should be sloped toward the water at a minimum grade of 1% and a maximum grade of 3%. The crossfall on the pathway is provided to promote the shedding of water deposited in this area by waves breaking over the wall crest.

Geotextile fabric layers are incorporated for two different purposes in different parts of the structure. One layer of geotextile should be laid from the toe of the excavation up along the trimmed face of the bank. This layer should be returned forward from the rear of the trench to under the pathway at the top of the structure. This layer is intended to prevent the migration of fine soil particles from surrounding areas into the graded backfill. It is necessary to prevent the migration of soil fines into the graded backfill to optimise free drainage of the area behind the wall. A second layer of geotextile fabric should overlay the first layer in the toe of the structure. This second layer should be used to encapsulate the gravel underfooting to be formed under the wall. The encapsulated underfooting should then retain the first layer of geotextile fabric as well as providing load spreading at block joints.

Omission of the concrete pathway from the rear of the structure at this site would expose the structure to the risk of significant scour behind the wall as happened previously with the timber log retaining wall at this site. While it is known that the waves will plunge over the wall crest as they break, estimation of the point of contact between the plunging wave and the ground surface behind the wall would require detailed physical modelling which is beyond the scope of this study. For this reason the width of the concrete pathway armour is based on engineering judgement of the likely plunge zone of the design waves and is intended to inhibit scour rather than totally prevent it.

Suitable provision should be made for the stormwater drainage pipeline at about the mid point of the site. The pipeline should either be provided with a precast headwall incorporating an apron and wingwalls to which the sandstone wall may be returned or alternatively should be provided with a flexible joint and projected at least 200mm through the sandstone wall with rip-rap scour protection at the outlet.

Notwithstanding the aspects of this design to limit damage from wave attack, scouring damage will occur from time to time and major repair of the terrace area behind the seawall will be necessary following such storm events.
Typical foreshore looking north

Existing foreshore protection looking south beyond northern end of site
Seawall to comprise of large rough cut sandstone blocks (0.5 tonne to 3 tonne) keyed in random pattern and dry grouted.

Seawall crest at 110 AHD
Larger blocks used for bottom row of seawall with weep holes provided by spacing blocks with 20 to 50mm gaps

Existing rocky/sandy slope
Sandstone footing at -0.50 AHD or deeper
Seawall toe trenched into sandy bed
Gravel underfooting at -0.80 AHD or deeper

Geotextile fabric sock encapsulating the gravel underfooting

NOTES
1 This design profile is typical for this foreshore section and may require appropriate on-site modification
2 Figure 11-1 needs to be read in conjunction with the text for Tascott
3 IMPORTANT - As indicated in Coffey Partners Report No G0625/1-AB the footing should be 1m wide with embedment of at least 0.5m
4 Council accepts that crest of this seawall will be overtopped on regular basis by wave action breaking over the wall causing scouring behind the wall. High tides more extreme than the 10 year ARI event will cause inundation irrespective of wave conditions
5 Gravel underfooting for load spreading to minimise differential settlement at joints between blocks in the bottom course
12. Yattalunga

12.1 Present Condition

Yattalunga lies within an embayment of Brisbane Water with Green Point to the north west and Saratoga to the south west. The embayment has a basically north western exposure with an effective fetch length of 3000m. For this fetch the 5 year ARI design wave is a 0.7m wave which at high water would reach the foreshore unbroken. The 5 year ARI design water level at this site is 1.06m AHD.

There is a reserve along the foreshore at this site which is generally about 20m wide with an elevation of about 1.1m AHD. The foreshore is depicted in Plate 12. At the northern end the reserve adjoins residential properties which front on to Mundoora Avenue. Further south the reserve adjoins properties with legal frontage but no practical access to Davistown Road. A sealed road which is a continuation of Mundoora Avenue has been constructed within the reserve to provide vehicular access for the residential properties in this vicinity, which are oriented to overlook Brisbane Water.

Dense stands of mangroves screen the foreshores to both the north and south of this site and no foreshore erosion is evident in these areas. The flat intertidal beach in front of the residential properties which has a general slope of about 1:100 or flatter, features no screening mangroves which is undoubtedly the reason why this section of foreshore is currently subject to erosion. The intertidal flats at this site extend about 100m out from the foreshore escarpment with the existing escarpment around 0.7m high. A long timber jetty extends out from the foreshore to the edge of the intertidal flats where a swimming enclosure is located at the edge of the drop off into deeper water.

The erosion of the foreshore at this site has already undercut several mature casuarinas leading to collapse of some and placing others under threat. The foreshore is lined with previous unsuccessful ad hoc attempts at protection using dead trees, building rubble and sandstone rip-rap tipped intermittently along the foreshore erosion escarpment.

Coffey were retained by GCC to conduct sub-surface investigations at the site consisting of excavating, logging and sampling of six hand auger holes of varying depths at approximately 50m intervals and conducting eleven dynamic cone penetrometer tests at approximately 25m intervals. Based on their investigations they concluded that the generalised sub-surface profile at the proposed seawall site at Yattalunga is summarised as follows.

**TOPSOIL**

(CL) Sandy Clay, low to medium plasticity, brown, some roots, observed unit thickness varied from 0 to 0.3m, overlying
FILL Sand and Silty Clay, some sandstone fragments, sheet steel and other debris, fill was observed at various locations over the length investigated, observed unit thickness varied from 0 to 0.5m, overlying

ALLUVIUM (SP) Sandy fine to medium grained, brown and orange-brown, medium dense to dense, observed unit thickness varied from 0.9m to 1.4m, overlying

ALLUVIUM/RESIDUAL (SL/CH) Sandy Clay, medium to high plasticity, light grey, very stiff to hard, unit was observed at depths of 1.5m to 1.6m

Coffey assessed that the allowable bearing capacity at the Yattalunga site is 150kPa

12.2 Outline of Options

There is a range of options for foreshore treatment at this site, which while technically acceptable from a foreshore protection viewpoint, are unlikely to be acceptable to the local residents, as they would restrict both the available views and water access from the reserve. These options include regeneration of the screening mangroves which it is believed were previously removed, sand nourishment including formation of a stabilised dune and gravel nourishment to form a cobble beach and dune.

The local residents have expressed a preference for the foreshore protection at the site to take the form of a low sandstone block wall along the entire frontage of the reserve, being returned into the mangroves at each end. GCC has endorsed a sandstone wall as an appropriate foreshore treatment at this site but would restrict the extent of the wall to that length of foreshore where the road is constructed on the reserve and trees are under threat. GCC also indicated that the wall crest level should not significantly exceed that of the terrace behind it.

12.3 Recommended Foreshore Treatment

Figure 12-1 shows a typical cross-section of the recommended foreshore treatment for Yattalunga. This diagram needs to be read in conjunction with this text. It should be noted that this design is for a typical cross-section and may require appropriate on-site modification which should be directed and supervised by experienced GCC staff.

The recommended treatment at this site is a sandstone block seawall, comprising large rough-cut sandstock blocks varying in size from say 0.5 tonne to 3.0 tonne. The blocks are to be keyed in a random pattern and dry grouted. The larger blocks should be placed at the base with weep holes provided by spacing blocks with a 20 to 50 mm gap. It is proposed to shape some of the sandstone rocks so as to provide the 1.0m wide footing required.

The seawall toe should be trenched into the sand to a depth of -0.80m AHD, with the rear face of the excavation trimmed up to the level of the terrace as shown in Figure 12.1. A gravel under-footing 0.3m thick encapsulated in geotextile fabric should be placed in this trench to provide load spreading at the joints between boulders in the footing course and minimise differential settlements. This will bring it up to the footing level of -0.5m AHD to ensure 0.5m embedment while allowing for scour at the toe of the wall.
The GCC directive for this site that the crest level of the structure be limited effectively means adopting a seawall design which will be regularly overtopped. The design has however been developed so as to limit both the incidence of tidal inundation of the reserve and the scour damage behind the structure caused by waves breaking over it. The crest level has been set at 1.10m AHD which corresponds to a design still water level with a 10 year ARI. However, some overtopping would be expected more frequently due to wave action coinciding with peak water levels of lesser ARIs. The design is based on the wave breaking when it reaches the front face of the wall and hence plunging over the top of the wall. The proposed concrete pathway at the rear face of the wall/terrace is an integral part of the design and is intended to armour this part of the wall to limit scouring of the graded backfill. The pathway should be sloped toward the water at a minimum grade of 1% and a maximum grade of 3%. The crossfall on the pathway is provided to promote the shedding of water deposited in this area by waves breaking over the wall crest.

Geotextile fabric layers are incorporated for two different purposes in different parts of the structure. One layer of geotextile should be laid from the toe of the excavation up along the trimmed face of the bank. This layer should be returned forward from the rear of the trench to under the pathway at the top of the structure. This layer is intended to prevent the migration of fine soil particles from surrounding areas into the graded backfill. It is necessary to prevent the migration of soil fines into the graded backfill to optimise free drainage of the area behind the wall. A second layer of geotextile fabric should overlay the first layer in the toe of the structure. This second layer should be used to encapsulate the gravel underfooting to be formed under the wall. The encapsulated underfooting should then retain the first layer of geotextile fabric as well as providing load spreading at block joints.

Omission of the concrete pathway from the rear of the structure at this site would expose the structure to the risk of significant scour behind the wall as happened previously with the timber log retaining wall at this site. While it is known that the waves will plunge over the wall crest as they break, estimation of the point of contact between the plunging wave and the ground surface behind the wall would require detailed physical modelling which is beyond the scope of this study. For this reason the width of the concrete pathway armour is based on engineering judgement of the likely plunge zone of the design waves and is intended to inhibit scour rather than totally prevent it. Notwithstanding the aspects of this design to limit damage from wave attack, scouring damage will occur from time to time and major repair of the terrace area behind the seawall will be necessary following such storm events.

In relation to this wall design the Coffey report makes the following observations: "It is understood that the proposed wall is to be constructed of rough cut sandstone and railway ballast. It is further understood from discussions with the wall designers that the maximum wall height is to be approximately 1.5m and it is expected that the wall will be occasionally overtopped. It is anticipated that the wall will have an embedment of approximately 0.5m with a height of about 1m.

Given the proposed wall heights and the anticipated unit weights of the construction materials, it is assessed that the required allowable bearing pressure to support the wall is approximately 40kPa."

The overall drainage from the Reserve needs to be assessed and accommodated by GCC in association with this seawall design.
The alignment of the seawall needs to be determined on-site but in general should follow the line of the existing foreshore escarpment. The north eastern end of the seawall should terminate in the general vicinity of Mundoora Avenue at a location to be determined on site. At this point, the wall should be returned for a short distance (say 2 metres) into the reserve terrace, if no existing suitably armoured foreshore feature is available for use as a terminus. The south western end of the wall should be returned to the existing rock protection of the outlet of the 900mm diameter stormwater pipeline.

It is anticipated that there will be some loss of sand from the slope in front of the (near) vertical seawall, due to the higher reflected wave energy. An allowance for this anticipated loss has been made in determining the levels of both the footing and gravel underfooting of the wall.
Typical foreshore looking north, despoiled by building rubble used in an attempt to avert foreshore erosion

Typical foreshore looking south, note loss of mature trees
NOTES

1. This design profile is typical for this foreshore section and may require appropriate on-site modification.

2. Figure 12-1 needs to be read in conjunction with the text for Yattalunga.

3. IMPORTANT - As indicated in Coffey Partners Report No G0525/1-AB, the footing should be 1.0m wide with embedment of at least 0.5m.

4. Council accepts that crest of this seawall will be overtopped on regular basis by wave action breaking over the wall causing scouring behind the wall. High tides more extreme than the 10 year ARI event will cause inundation irrespective of wave conditions.

5. Gravel underfooting for load spreading to minimize differential settlement at joints between blocks in the bottom course.
APPENDIX A

Coffey Partners International Pty Ltd Geotechnical Report of selected sites of proposed seawalls on Brisbane Water Foreshore
ATTENTION MR CHRIS ARGAE

Dear Sir

RE PROPOSED SEA WALLS, BRISBANE WATER - GEOTECHNICAL INVESTIGATION

Please find enclosed our report describing geotechnical studies undertaken at four proposed sea wall sites in Brisbane Water. The report describes surface and subsurface conditions encountered at the sites and gives recommendations on allowable bearing pressures for the proposed sea walls.

Should you have any queries please do not hesitate to contact the undersigned in our Gosford office.

For and on behalf of
COFFEY PARTNERS INTERNATIONAL PTY LTD

B. A STEPHENS
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Important Information About Your Geotechnical Engineering Report

APPENDIX A - Summary Logs

APPENDIX B - Dynamic Penetrometer Test Results
1.0 INTRODUCTION

At the request of Mr Chris Argaet of Gosford City Council, Coffey Partners International Pty Ltd have carried out geotechnical studies at four proposed sites for sea walls within Brisbane Water. The purpose of these studies was to assess foundation conditions for the proposed walls and estimate allowable bearing pressures for the design of the sea walls.

The proposed sea wall locations investigated were at Bowden Road, Woy Woy Pier Avenue, Saratoga, Koolewong Foreshore Reserve, Koolewong and Mundoora Avenue, Yattalunga. This work was carried out generally in accordance with our proposal No GOP367/1 dated 11th December 1992.

It is understood the proposed sea walls are to be approximately 1.0m to 2.5m in height and are to be constructed from blocky sandstone and railway ballast. The walls are to be located such that cut and fill quantities are minimised.

Presently two alternative designs are being considered for the sea walls. The first utilises a concrete strip footing to support the wall, comprising mortared rough cut sandstone. The second design involves using large sandstone blocks placed in trenches to found the sea wall. Geofabric would be placed under the sandstone blocks and wall backfill.

Preliminary drawings provided by Manly Hydraulics Laboratory indicate that the wall is to comprise rough cut sandstone blocks 0.5 to 3.0 tonne. These rocks are to be keyed in a random pattern and dry grouted. It is also noted that the design assumes that the crest of the sea wall will occasionally be overtopped.

2.0 FIELD WORK

Field work comprised the excavating, logging and sampling of hand auger holes at approximately 50m centres at the locations of the proposed sea walls. Dynamic Penetrometer tests were carried out at approximately 25m intervals.

Summary logs of the hand auger holes are given in Appendix A. Results of the Dynamic Penetrometer tests are given in Appendix B. The locations of the hand auger holes and Dynamic Penetrometer tests are given in Drawing Nos. G0625/1-1 to 4. These were obtained by making measurements relative to features shown.

3.0 SITE CONDITIONS

3.1 Surface

3.1.1 Woy Woy

The area investigated for the proposed sea wall at Woy Woy is located on the foreshore between Bowden Road, Woy Woy and an existing boat shed approximately 450m to the north. The width of the foreshore reserve at this location was generally 30m, however this width did vary with location.
Numerous small sea walls were observed within the area investigated. These walls were predominantly short sections of walls, and were observed to be constructed from various materials such as brick, concrete, ripped sandstone and timber. It is understood from discussions with the Client that these walls were constructed by the residents adjacent to the foreshore. Erosion of foreshore areas was observed in areas without sea walls.

The foreshore in the area investigated was observed to be vegetated by a thick grass cover. Numerous large trees were also located in the foreshore reserve, generally located away from the shoreline.

Some mangrove plants were observed near Bowden Road. No other mangroves were observed in the area of the investigation.

3.1.2 Yattalunga

The area of the proposed sea wall at Yattalunga is located off Mundoora Avenue, Yattalunga. The area of foreshore investigated was approximately 150m to the north and south of the public wharf located off Mundoora Avenue.

It is understood from discussions with the Client that the area has undergone significant erosion due to removal of mangroves from the foreshore. Attempts at arresting the erosion have been made by the dumping of ripped sandstone along the shore. This appears to have had limited success.

Mangroves are located to the north and south of the area investigated. Only small mangroves (<1m) were observed in the foreshore of the area investigated.

3.1.3 Koolewong

The proposed sea wall to be located at Koolewong Foreshore reserve is located between an existing dry rock retaining wall to the north and a sandstone outcrop to the south. The site is adjacent to Brisbane Water Drive and comprises a fenced off reserve with a shelter and toilet block.

The remains of an old log retaining wall was located on the shoreline of the park. The wall was observed to be approximately 0.6m high.

The reserve was observed to have a good cover of grass with numerous trees along the foreshore. Some small mangroves were observed off the foreshore.
3.1.4 Saratoga

The site of the proposed sea wall at Saratoga is located to the west of Saratoga Oval, Saratoga. The proposed wall is to be built between existing mangroves to the south and an area of outcropping sandstone to the north.

Ripped sandstone appears to have been placed in the foreshore adjacent to Saratoga Oval, with some signs of erosion adjacent to an existing boat launching ramp.

The area was observed to have a good cover of grass near the foreshore. Some trees were located at a distance from the shoreline.

3.2 Subsurface

3.2.1 Woy Woy

The generalised subsurface profile at the Woy Woy site is summarised as follows:

- **TOPSOIL/FILL.** (CL) Sandy Clay, low plasticity, brown, some silt, wet, observed unit thickness varied from 0 to 0.6m, overlying
- **ALLUVIUM** (SP) Sand, fine to medium grained, grey and brown, some sand and sandy clay zones, loose to dense generally medium dense, unit observed to depths of 1.8m

3.2.2 Yattalunga

The generalised subsurface profile at the proposed sea wall site at Yattalunga is summarised as follows:

- **TOPSOIL.** (CL) Sandy Clay, low to medium plasticity, brown, some roots, observed unit thickness varied from 0 to 0.3m, overlying
- **FILL** Sand and Silty Clay, some sandstone fragments, sheet steel and other debris, fill was observed at various locations over the length investigated, observed unit thickness varied from 0 to 0.5m, overlying
- **ALLUVIUM** (SP) Sandy, fine to medium grained, brown and orange-brown, medium dense to dense, observed unit thickness varied from 0.9m to 1.4m, overlying
ALLUVIUM/RESIDUAL. (CL/CH) Sandy Clay, medium to high plasticity
light grey, very stiff to hard, unit was
observed at depths of 1.5m to 1.6m

3 2.3 Koolewong

The generalised subsurface profile at the Koolewong sea wall site
is summarised as follows

TOPSOIL/FILL (CL) Sandy Clay, low plasticity, bricks and
sandstone observed in matrix, some peaty zones
observed unit thickness varied from 0.3m to
0.6m overlying

ALLUVIUM (SP) Sand, fine to medium grained, light grey
trace of fines, medium dense to dense, observed
unit thickness varied from 0.9m to 1.2m
overlying

ALLUVIUM/RESIDUAL. (SC) Clayey Sand, fine to medium grained, light
grey, medium dense to dense, unit was observed
to depths of 1.7m

3 2.4 Saratoga

Based on the results of the hand auger holes the subsurface
profile at the Saratoga site may be summarised as follows

TOPSOIL (SP) Silty Sand, fine grained, brown, some
roots, observed unit thickness varied from 0 to
0.2m, overlying

ALLUVIUM (SP) Sand, fine grained, brown and orange-brown
some silt, some shells, observed unit thickness
varied from 0.7m to 1.6m, overlying

ALLUVIUM/RESIDUAL. (SC) Clayey Sand, fine grained, light grey,
moist, dense, unit was observed between 0.7m to
greater than 1.8m depth

4.0 DISCUSSION AND RECOMMENDATIONS

4.1 Allowable Bearing Capacity

It is understood that the proposed wall is to be constructed of rough cut
sandstone and railway ballast. It is further understood from discussions with
the wall designers that the maximum wall height is to be approximately 1.5m
and it is expected that the wall will be occasionally overtopped. It is
anticipated that the wall will have an embedment of approximately 0.5m with a
height of about 1m.
Given the proposed wall heights and the anticipated unit weights of the construction materials, it is assessed that the required allowable bearing pressure to support the wall is approximately 40kPa.

Based on the results of the hand auger holes and Dynamic Penetrometer testing, it is recommended that the allowable bearing pressures given in Table 1 be adopted for design of the sea walls. The allowable bearing capacities given in Table 1 have been calculated assuming a footing width of 1.0m and embedment of at least 0.5m.

It should be noted that the bearing capacity of sand footings is dependent on factors such as embedment depth, length and width of the footing. Variations in these parameters may result in a change in the allowable bearing pressures of the footings. Please contact this office if other footing dimensions are to be used.

<table>
<thead>
<tr>
<th>Site</th>
<th>Assessed Allowable Bearing Pressure (KPa)</th>
<th>(see text)</th>
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</thead>
<tbody>
<tr>
<td>Woy Woy</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Yattalunga</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>Koolewong</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>Saratoga</td>
<td>150</td>
<td></td>
</tr>
</tbody>
</table>

### 4.2 Construction

It should be noted that the sites investigated are susceptible to tidal fluctuations which may make excavation of the footings difficult. From the investigation it was observed that the alluvial sands at the anticipated footing depth were generally saturated, even at low tide.

It is assessed that the saturated sands and tidal fluctuations present at the sites may make the construction of the proposed concrete footings difficult. The alternative of using a geofabric and large rough cut sandstone blocks as a footing appears to be a more practical solution given the observed site conditions.

Based on the results of the Dynamic Penetrometer testing it is anticipated that isolated pockets of loose alluvial materials may be encountered at locations along the walls. These sections may be treated by either over excavating to found on medium dense materials below, or else compacting the insitu sands to a medium dense condition.

For and on behalf of
COFFEY PARTNERS INTERNATIONAL PTY LTD
IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

As the client of a consulting geotechnical engineer you should know that site subsurface conditions cause more construction problems than any other factor. ASFE/The Association of Engineering Firms Practicing in the Geosciences offers the following suggestions and observations to help you manage your risks.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

Your geotechnical engineering report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. These factors typically include the general nature of the structure involved, its size and configuration, the location of the structure on the site, other improvements such as access roads, parking lots, and underground utilities, and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask your geotechnical engineer to evaluate how factors that change subsequent to the date of the report may affect the report's recommendations.

Unless your geotechnical engineer indicates otherwise, do not use your geotechnical engineering report:

• when the nature of the proposed structure is changed for example, if an office building will be erected instead of a parking garage, or a refrigerated warehouse will be built instead of an unrefrigerated one,
• when the size, elevation, or configuration of the proposed structure is altered,
• when the location or orientation of the proposed structure is modified,
• when there is a change of ownership, or,
• for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems that may occur if they are not consulted after factors considered in their report's development have changed.

SUBSURFACE CONDITIONS CAN CHANGE

A geotechnical engineering report is based on conditions that existed at the time of subsurface exploration. Do not base construction decisions on a geotechnical engineering report whose adequacy may have been affected by time. Speak with your geotechnical consultant to learn if additional tests are advisable before construction starts. Note too that additional tests may be required when subsurface conditions are affected by construction operations or adjacent to the site, or by natural events such as floods, earthquakes, or ground water fluctuations. Keep your geotechnical consultant apprised of any such events.

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL JUDGMENTS

Site exploration identifies actual subsurface conditions only at those points where samples are taken. The data were extrapolated by your geotechnical engineer who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your geotechnical engineer can work together to help minimize their impact. Retaining your geotechnical engineer to observe construction can be particularly beneficial in this respect.

A REPORT'S RECOMMENDATIONS CAN ONLY BE PRELIMINARY

The construction recommendations included in your geotechnical engineer's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Because actual subsurface conditions can be discerned only during earthwork, you should retain your geotechnical engineer to observe actual conditions and to finalize recommendations. Only the geotechnical engineer who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations are valid and whether or not the contractor is abiding by applicable recommendations. The geotechnical engineer who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Consulting geotechnical engineers prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your geotechnical engineer prepared your report expressly for you and expressly for purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the geotechnical engineer. No party should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

GEOENVIRONMENTAL CONCERNS ARE NOT AT ISSUE

Your geotechnical engineering report is not likely to relate any findings, conclusions, or recommendations...
about the potential for hazardous materials existing at the site. The equipment, techniques, and personnel used to perform a geoenvironmental exploration differ substantially from those applied in geotechnical engineering. Contamination can create major risks. If you have no information about the potential for your site being contaminated, you are advised to speak with your geotechnical consultant for information relating to geoenvironmental issues.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid misinterpretations, retain your geotechnical engineer to work with other project design professionals who are affected by the geotechnical report. Have your geotechnical engineer explain report implications to design professionals affected by them, and then review those design professionals’ plans and specifications to see how they have incorporated geotechnical factors. Although certain other design professionals may be familiar with geotechnical concerns, none knows as much about them as a competent geotechnical engineer.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE REPORT

Geotechnical engineers develop final boring logs based upon their interpretation of the field logs (assembled by site personnel) and laboratory evaluation of field samples. Geotechnical engineers customarily include only final boring logs in their reports. Final boring logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, give contractors ready access to the complete geotechnical engineering report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report’s limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared and that developing construction cost estimates was not one of the specific purposes for which it was prepared. In other words, while a contractor may gain important knowledge from a report prepared for another party, the contractor would be well-advised to discuss the report with your geotechnical engineer and to perform the additional or alternative work that the contractor believes may be needed to obtain the data specifically appropriate for construction cost estimating purposes. Some clients believe that it is unwise or unnecessary to give contractors access to their geotechnical engineering reports because they hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems. It also helps reduce the adversarial attitudes that can aggravate problems to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical engineers. To help prevent this problem, geotechnical engineers have developed a number of clauses for use in their contracts, reports, and other documents. Responsibility clauses are not exculpatory clauses designed to transfer geotechnical engineers’ liabilities to other parties. Instead, they are definitive clauses that identify where geotechnical engineers’ responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report. Read them closely. Your geotechnical engineer will be pleased to give you full and frank answers to any questions.

RELY ON THE GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE

Most ASFE-member consulting geotechnical engineering firms are familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a construction project. From design through construction, speak with your geotechnical engineer, not only about geotechnical issues but others as well, to learn about approaches that may be of genuine benefit. You may also wish to obtain certain ASFE publications. Contact a member of ASFE for a complimentary directory of ASFE publications.

* For further information on this aspect reference should be made to "Guidelines for the Provision of Geotechnical Information in Construction Contracts" published by the Institution of Engineers Australia, National Headquarters, Canberra 1987.

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APPENDIX A

Summary Logs

WOY WOY

HW1
0-0 5m TOPSOIL/FILL Sandy Clay, low plasticity, brown, some silt, wet
0 5-1 3m ALLUVIUM Sand, fine to medium grained, brown, wet
1 3-1 7m ALLUVIUM Sandy Clay, low plasticity, brown, wet
1 7-1.9m ALLUVIUM Sand, fine to medium grained, grey

END HAND AUGER HOLE AT HW1 AT 1.9M

HW2
(0.5m below W3 in front of wall)
0-1 0m ALLUVIUM Clayey Sand, fine grained, brown, low plasticity clay
1 0-1 7m ALLUVIUM Clayey Sand, fine to medium grained, grey, wet

END HAND AUGER HOLE HW2 AT 1.7M, HOLE COLLAPSING

HW3
0-1.4m ALLUVIUM: Sand, fine to medium grained, brown, wet

END HAND AUGER HOLE HW3 AT 1.4M, NO RECOVERY.

HW4
0-1.4m ALLUVIUM. Sand, fine to medium grained, brown, wet, some clayey sand zones

END HAND AUGER HOLE HW4 AT 1.4M, NO RECOVERY.

HW5
0-1.3m ALLUVIUM: Sand, fine to medium grained, grey, wet, trace clay

END HAND AUGER HOLE HW5 AT 1.3M, HOLE COLLAPSING.

HW6
0-1.3m ALLUVIUM: Sand, fine to medium grained, grey

END HAND AUGER HOLE HW6 AT 1.3M, HOLE COLLAPSING.

HW7
0-1.2m ALLUVIUM: Sand, fine to medium grained, grey

END HAND AUGER HOLE HW7 AT 1.2M, HOLE COLLAPSING.
HW8 0-0.5m ALLUVIUM Clayey Sand, fine grained, brown

END HAND AUGER HOLE HW8 AT 1.6M, HOLE COLLAPSING.

HW9 0-1 7m ALLUVIUM Sand, fine to medium grained, grey-brown, trace shells, trace fines

END HAND AUGER HOLE HW9 AT 1.7M.

YATTALUNGA

HY1 0-0.2m TOPSOIL Sandy Clay, medium plasticity, brown, wet, some roots
0 2-0 6m FILL Sand, fine to medium grained, brown and light brown, some sandy clay and sandstone fragments
0.6-1 5m ALLUVIUM Sand, fine grained, brown
1 5-1 7m ALLUVIUM/RESIDUAL Clayey Sand, fine grained, light grey

END HAND AUGER HOLE HY1 AT 1.7M.

HY2 0-0.6m FILL: Sandy Clay, low plasticity, brown, some sand zones, some sandstone and brick fragments
0.6-1 5m ALLUVIUM Sand, fine grained, brown and orange-brown

END HAND AUGER HOLE HY2 AT 1.5M, HOLE COLLAPSING (Numerous pieces of sandstone fill in area, 3 attempts to get down).

HY3 0-0.3m TOPSOIL Sandy Clay, low plasticity, brown, abundant roots
0.3-1.7m ALLUVIUM Sandy, fine to medium grained, orange-brown

END HAND AUGER HOLE HY3 AT 1.7M.

HY4 0-0.5m FILL Silty Clay, low plasticity, brown, some tin and bricks, some sand zones
0.5-0 7m ALLUVIUM Silty Clay, low plasticity, brown
0 7-1.6m ALLUVIUM Sand, fine to medium grained, light grey, occasional trace of peat at depth
1 6-1.7m ALLUVIUM/RESIDUAL Sandy Clay, medium to high plasticity, light grey

END HAND AUGER HOLE HY4 AT 1.7M.
HY5 0-1 3m ALLUVIUM Sand, fine grained, white and grey, some silt, wet
1 3-1 4m ALLUVIUM/RESIDUAL: Clayey Sand, fine grained, light grey, moist

END HAND AUGER HOLE HY5 AT 1.4M, REFUSAL OF HAND AUGER.

HY6 0-0 4m ALLUVIUM Silty Sand, fine grained, brown
0 4-1 5m ALLUVIUM: Sand; fine grained, grey
1 5-1 6m ALLUVIUM/RESIDUAL: Sandy Clay, medium plasticity, light grey

END HAND AUGER HOLE HY6 AT 1.6M.

KOOLEWONG

HK1 0-0 3m FILL Bricks, sandstone and other deleterious materials, sandy clay matrix, some tree roots
0 3-0 6m ALLUVIUM: Clayey Sand, fine grained, brown, wet
0 6-1 3m ALLUVIUM: Sand, fine to medium grained, light brown
1 3-1 7m ALLUVIUM/RESIDUAL: Clayey Sand, fine to medium grained, light grey, medium dense to dense

END HAND AUGER HOLE HK1 AT 1.7M, HOLE COLLAPSING @ 1M.

HK2 0-0 3m TOPSOIL: Sandy Clay, low plasticity, dark brown, some peaty zones
0 3-1.2m ALLUVIUM: Sand, fine to medium grained, light grey, trace of clay fines
1 2-1.5m ALLUVIUM/RESIDUAL: Clayey Sand, fine to medium grained, light grey, moist

END HAND AUGER HOLE HK2 AT 1.5M.

HK3 0-0 4m TOPSOIL FILL. Sandy Clay, low plasticity, dark brown, some tree roots, some peaty clay zones
0 4-1 6m ALLUVIUM: Sand, fine to medium grained, light grey, wet

END HAND AUGER HOLE HK3 AT 1.6M.
HK4 0-0.6m FILL Silty Sandy Clay, low plasticity, dark brown, numerous large sandstone fragments at about 0.5m

END HAND AUGER HOLE HK4 AT 0.6M, REFUSAL ON LARGE SANDSTONE

SARATOGA

HS1 0-0.2m TOPSOIL Silty Sand, fine grained, brown, some roots
0.2-1.8m ALLUVIUM Sand, fine grained, brown, some silt, wet

END HAND AUGER HOLE HS1 AT 1.8M, NO RECOVERY

HS2 0-1.3m ALLUVIUM Sand, fine grained, light brown and grey, grey content increasing with depth, some silt, numerous shells at surface, wet
1.3-1.5m ALLUVIUM/RESIDUAL Clayey Sand, fine grained, light grey, moist

END HAND AUGER HOLE HS2 AT 1.5M.

HS3 0-0.7m ALLUVIUM Sand, fine grained, brown and orange-brown
0.7-1.0m ALLUVIUM/RESIDUAL Clayey Sand, fine grained, light grey

END HAND AUGER HOLE HS3 AT 1.0M.
test results

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## Test Results

**Principal Project Location:**
- Gosford City Council
- Brisbane Water Sea Walls
- Woy Woy

**Test Procedure:** Dynamic Cone Penetrometer

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**Date:** 22/12/92

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**Phone:** 

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Coffey Partners International Pty Ltd
ACN 003 692 019

 Consulting engineers in the geotechnical sciences
Incorporated in NSW

46 Hill Street, Gosford
Phone 432 583
## Test Results

**Principal Project:** Gosford City Council  
**Location:** Brisbane Water Sea Walls  
**Location:** Yatalaunga  
**Date:** 22/12/92  
**Ref:** 00625/1

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**Test Procedure:** Dynamic Cone Penetrometer  
**Reading Recorded In:** B105 P5% 50%
**TEST RESULTS**

**Project:**
GOSFORD CITY COUNCIL

**Location:**
BRISBANE WATER SEA WALLS
YATMALUNGA

**Date:** 22/12/92

**Instrument:** DYNAMIC CONE PENETROMETER

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### Test Procedure: DYNAMIC CONE PENETROMETER

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APPENDIX B

"A Guide To Mangrove Transplanting"

Published by State Pollution Control Commission, Concord Municipal Council and Division of Fisheries, Department of Agriculture
A Guide to
Mangrove Transplanting

State Pollution Control Commission
Concord Municipal Council
Division of Fisheries, Department of Agriculture
1 Introduction

In October 1978 Concord Municipal Council resolved to carry out a mangrove planting programme in its municipality along the foreshores of the Parramatta River.

The principal objective was to revegetate selected sections of the foreshores with mangroves, replacing the important mangrove habitat lost through over a century of riverside development. A secondary objective was to complement the council’s ongoing programme of foreshore beautification, especially along the frontages of industrial estates.

At an early stage the council consulted with the Maritime Services Board and State Fisheries, and both departments approved the council’s proposed programme. State Fisheries was particularly interested in the rehabilitation of this mangrove habitat, both in terms of the possible benefits to fish in Sydney Harbour and in terms of broader applications to estuaries elsewhere in New South Wales.

The State Pollution Control Commission, which is responsible for co-ordinating the activities of all public authorities in the fields of pollution control and environmental protection, also became involved in the programme. In line with its responsibilities in the administration of the Clean Waters Act, the SPCC was particularly interested in the potential benefits for water quality in New South Wales estuaries.

On this basis a co-operative programme was initiated, and this booklet is one of the results.
2 Why mangroves are important

Mangroves are important for a number of reasons:

- They are major producers of organic material and may have a special role in supporting estuarine fisheries (finfish, crustaceans and shellfish).
- They are involved in nutrient recycling.
- They help to reduce water pollution.
- They provide shelter, refuge and food for many forms of wildlife.
- They help prevent bank erosion and provide protection from storm surge.
- They act as visual screens along industrial foreshores, improving the amenity of the waterway.
- Mangrove habitats act as important nursery areas for many economically important (commercial and angling) fish species.

The dominant economically important fish species found in mangrove habitats include bream (*Acanthopagrus* spp), blackfish (*Girella tricuspidata*), mullet (*Mugilidae* spp), flathead (*Platycephalus fuscus*), silver biddy (*Gerres ovatus*) and whiting (*Sillago* spp).

These species are present in the mangrove habitat both as temporary residents when juveniles, principally in autumn and winter, and as transient residents when adults. The major reasons for the selection of this habitat by juveniles appear to be shelter and the availability of food. Flat-tailed mullet feed at high tide on the myriad insect life amongst the mangrove peg roots (pneumatophores), while bream, silver biddy and whiting feed on aggregations of microcrustaceans and blackfish feed on algae attached to the peg roots and exposed roots on the creek banks.

It is not possible to place an exact figure on the monetary value of the mangrove habitat. However, in the ten years prior to 1972 the estuarine-dependent fish catch in New South Wales, including fish from mangrove habitats, was about 66 per cent by weight and over 70 per cent by value of the total fish catch in the State. The NSW fish catch in 1981/82 was valued at $72.5 million, and the estimated gross value of production from fisheries throughout Australia in 1981/82 was $396 million. In addition, the important recreational fishery in Australia makes a contribution to the economy possibly equal to or even greater than that of the commercial fishing industry, when one considers both sales and associated service industries.

3 Mangrove species

Within the Sydney region two species of mangrove are commonly found:

- The grey mangrove (*Avicennia marina* (Forsk) Vierh.)
- The river mangrove (*Aegiceras corniculatum* (L) Blanco).

The grey mangrove is a medium-sized tree, or occasionally a shrub, with a well-defined trunk and numerous vertical peg roots (pneumatophores).
The leaves are green above and greyish below, are ovate in shape with an acute apex, and occur opposite one another on the stems. The seed is viviparous (i.e., it germinates on the tree), and has a large pair of seed leaves (cotyledons) which give it the appearance of a large flat bean. Seed fall is normally in summer.

The river mangrove is generally a small shrub, often with several small trunks and no peg roots. The leaves are glossy green above and paler underneath, are ovate in shape but with a rounded apex, and occur alternately along the stem. The seed is small, curved and viviparous, and normally falls in late winter.

Both of these species have been successfully transplanted in the Sydney region. They are exceptionally adaptable to varying environmental conditions within estuaries. This adaptability is further shown by their ability to be grown as ornamental plants in pots. Transplanting success rates are improved, however, when a number of environmental parameters are taken into account.

4 Environmental considerations

Both mangrove species will grow in substrates ranging from gravel to fine mud or clay, with most mature forests occurring in areas of sandy mud. Frequently the substrate is anaerobic (lacking in oxygen). The two species of mangrove are also found throughout the salinity range from virtually freshwater to sea water. Both species occur together in suitable areas, although the grey mangrove prefers salinities close to sea water and the river mangrove prefers brackish water.

Before transplanting mangroves, the foreshore tidal plane must be considered. Trees can be found growing both terrestrially and fully submerged, but optimum growth and establishment success appear to occur just above the mid-tide level.

Another factor to consider is exposure to waves. It has been demonstrated that the major cause of transplanting failure is seedling damage caused by high wave action and associated flotsam and jetsam. Wave energy along exposed reclaimed foreshores fronted by retaining walls is dissipated over a short distance below the wall. This makes such foreshores an unstable environment in which to establish mangrove transplants. In these situations physical protection of the transplants is necessary (see section 5).
5 Methods of establishment

Both species produce large quantities of viviparous seeds which can be collected in the appropriate months (see section 3). The seeds can be easily transported and hand sown in selected locations by placing the sprouting seed 1 to 2 centimetres into the substrate. This technique is not suitable for locations where wave and tidal action can easily dislodge the seed, but is highly appropriate for increasing the density of existing mangrove stands, where the seeds can be lodged amongst the peg roots, or for sites which are well protected.

The most common transplanting method, however, is to use seedlings. Seedlings should be collected from large, mature mangrove stands where regular natural seeding occurs, so that their removal will not affect the viability of the forest. Within these mature forests most seedlings germinate, grow to about 0.6 metres, remain at this stage of development for up to five years and then die. This inhibition of growth appears to be related to competition for light and space in the understorey of mature forests. An estimate of seedling age can be made from the degree of woodiness (lignification) of the trunk base.

Seedlings chosen for transplanting should be about 0.5 metres long with a straight trunk. They should be no more than 18 months old and should have 6 to 10 leaves and no peg-root development. Collection and planting are best carried out during low tide.
Typical growth of a gray mangrove:

(a) Six months after being transplanted

(b) 2½ years after being transplanted
Removal of seedlings from the substrate is facilitated by using a length of hollow, 100 millimetre diameter PVC pipe as a corer pushed 20 to 25 centimetres into the substrate around the seedling. The pipe, containing the seedling and a "plug" of earth, is then removed intact. A little water poured gently down the pipe will help to shake the seedling and earth plug out of the corer.

During collection and transportation the seedling and earth plug should be kept moist in a container (e.g., a plastic bin), protected from direct sunlight and wind. To reduce the possibility of drying out, transplanting in summer is not recommended.

Investigations are currently in progress to design a corer which can be applied to large-scale transplanting (contact the persons listed in section 8 for further information).

When planting, a preparatory hole is dug and the earth plugs placed in position. Depending on the degree of exposure to wave action, each seedling may be tied to a garden stake using "budding" tape so that it remains in an upright position. Additional protection may be provided by the use of a shade-cloth fence or by piling rocks around the seedlings to prevent algae and flotsam and jetsam being left stranded on the seedlings by receding tides.

Best results are achieved by planting seedlings in close clumps at 20 to 25 centimetre intervals rather than by planting them in neatly spaced rows. Clumping offers the central plants additional protection.

The cost of seedling transplanting can be determined on the basis of a transplanting rate of 80 seedlings per two-person team per day. This includes collection, minimal transport time and planting. Transplanting success rates and individual tree growth rates are highly variable. As explained in section 4, they depend greatly on the areas into which the seedlings are transplanted.

Along the Parramatta River foreshores, for example, success rates after four years have ranged from 0 per cent (below exposed retaining walls with no additional protection given to transplants) to 45 per cent in more protected situations, with the greatest loss of seedlings occurring in the first two months after planting.

### 6 Typical applications

The possible situations in which transplanting of mangroves can be applied are many and varied. They range from culture in pots to the restoration of the degraded wetland areas which are so vitally important to estuarine ecosystems.

Examples of situations in which mangrove transplanting has been successful are:

- Along reclaimed foreshores formerly occupied by wetlands.
- As a fringing strip along retaining walls, where both screening for aesthetic reasons and habitat restoration are desirable. This is particularly relevant where "terrestrial" landscaping may be impractical (e.g., where buildings or other structures abut the foreshore).
On shallow areas created either by nature or by foreshore restoration following development. One example is at Chipping Norton in the Georges River, where ponds remaining after sand extraction have been redeveloped as a State Recreation Area with a variety of wildlife habitats, both terrestrial and aquatic.

As a requirement of habitat restoration following a development which involved some initial mangrove removal. An example is the foreshore-restoration works being undertaken at the Hawkesbury River following construction of the Sydney/Newcastle oil and gas pipeline.

7 Legal considerations
Mangroves in New South Wales are protected under the Fisheries and Oyster Farms Act 1935. This Act prohibits the cutting of mangroves on Crown Lands, which are defined so as to include any foreshore below the high-water mark, unless a permit has been obtained under the Act. Approval under the Act is therefore required to remove established seedlings and transplant them in another location.

In the case of lands vested in the Maritime Services Board, including Newcastle, Sydney, Botany Bay, and Port Kembla Harbours, approval under the Board’s Act is required before removal of mangroves can occur.

Advice on these various legal requirements may be obtained from the persons listed in section 8 below.

8 Further information
Additional advice on planting techniques and legal requirements may be obtained by contacting:
Mr P G Stewart,
Environmental Officer,
Concord Municipal Council,
Wellbank Street, Concord 2137.

Mr P Gibbs,
Environmental Studies Biologist
Division of Fisheries
Department of Agriculture, NSW
PO Box K220, Haymarket 2000.

The Secretary,
Coast and Wetlands Society,
PO Box A 225, Sydney South 2000.

General information may also be obtained from the
State Pollution Control Commission
157 Liverpool Street,
Sydney 2000.

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Look after our water