EAST HILLS
BANK EROSION STUDY
Georges River

June 1990
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FOREWORD

This report examines historical morphological changes in the Georges River adjacent to the East Hills Golf Course. It concentrates mainly on bank erosion over time with an assessment of future trends in river bank movement, and recommends possible foreshore protection measures to combat the problem.

The study has been undertaken by the Public Works Department for Bankstown City Council to assist in dealing with a development application for the redevelopment of East Hills Golf Course. The developer, Chiyoda Australia Pty Ltd, is funding this study with Freehill, Hollingdale and Page co-ordinating the redevelopment project until the granting of development consent.

This report was prepared by Estuary Management Section, Coast and Rivers Branch, 140 Phillip Street, Sydney.
SUMMARY

This report examines historical changes in the behaviour of a 2.5 km stretch of the Georges River adjacent to the East Hills Golf Course Redevelopment site. It makes a detailed examination of the rate and distribution of bank recession, and from these results makes an assessment regarding future trends in riverbank movements and recommends foreshore protection methods that may be used to arrest the erosion problem.

Erosion in this portion of the Georges River has been occurring over a long period and is the natural outcome of river migration on the alluvial floodplain of the valley. Since 1930 the extent of erosion has been up to 20m in places, primarily located at the outside of river bends. An examination of aerial photographs reveals fluvial geomorphological features such as scroll bars and abandoned meander patterns which show the river to have migrated hundreds of metres in the last thousands of years. As the river has tended to migrate across the floodplain in the past, so it will in the future unless prevented by structural intervention.

Recent rates of erosion between 1930 and 1988 have been accurately analysed by photogrammetric measurement from aerial photographs. In an effort to determine whether flooding was the principal erosion agent, aerial photographs were selected for analysis on the basis of bracketing periods of high and low flood incidence. Accordingly, photogrammetry was undertaken using 1930, 1949, 1970, 1978 and 1988 photography. Details of the movements of riverbank scarps since 1930 are presented as comparative overlays on Figure 3.6 Sheet A to D, while rates of erosion for various time periods are on Figure 3.8. Table 1 (over-page) summarises river behaviour from 1930 to 1988. Note that the golf course redevelopment site is on the “left bank” of the river.

Approximately 1.5 km of riverbanks in the study area have been affected to various degrees by piecemeal foreshore protection works, with the majority of these works situated on the right bank opposite the redevelopment site. The effect that these works have had on arresting erosion has varied from one location to another depending on the extent of protection. Apart from cases where the bank erosion rate has been altered by reclamation and/or foreshore protection works, the erosion rate has been generally consistent over the period of analysis, i.e., the last 60 years. Since boat traffic and dredging activities have only been factors operating over the last 40 years, this uniform erosion rate over the whole of the analysis period tends to confirm the view that the main cause of bank erosion is due to natural river migration. General experience suggests that flooding may be a contributing mechanism to bank erosion, however the results suggest that different periods of high and low flood incidence have had little effect on the rate of bank erosion.

Except for a section of river between km0 and km0.5 (see locations on Figure 2.3) the river channel adjacent to the study area has been very heavily dredged during...
the 1950's and 1960's. The river depths and cross sectional areas have tripled at some locations, with average river depth of around 3 to 4m in 1926, whereas today it averages around 6 to 7m with local holes up to a maximum of 10m.

Two reaches have been identified as experiencing bank erosion related to the development site, and are referred to as Sites 1 and 2 (Figure 2.2). The upstream Site 1 shows a relatively constant recession over the period 1930 to 1988 of between 0.2-0.25 m/year. The consistent rate over varying periods of high and low flood incidence indicates that natural river migration is the primary cause of bank erosion in this area. Downstream at Site 2, adjacent to the Golf Course Clubhouse, the results show a reduced rate and increased variability over time compared with Site 1. Rates have fluctuated over time but are generally in the range of 0.1-0.15 m/year for the last 60 years. The fluctuations may have largely been due to piecemeal bank protection and reclamation works adjacent to the riverbank.

### TABLE 1
**SUMMARY OF FORESHORE BEHAVIOUR: 1930 – 1988**

<table>
<thead>
<tr>
<th>Chainage 1 (km)</th>
<th>Left or 2 Right Bank</th>
<th>Scarp Movement 3 Since 1930 (m)</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 – 0.45 (Site 2)</td>
<td>L</td>
<td>-5 to -10m</td>
<td>Channel invert deep near bank. Erosion rate altered due to minor reclamation and protection works.</td>
</tr>
<tr>
<td>0 – 0.45</td>
<td>R</td>
<td>Stable</td>
<td>Heavy covering of mangroves.</td>
</tr>
<tr>
<td>0.45 – 1.0</td>
<td>L</td>
<td>No Scarp</td>
<td>Inside of bend. Heavy covering of mangroves.</td>
</tr>
<tr>
<td>0.45 – 1.0</td>
<td>R</td>
<td>-10 to -15m</td>
<td>Erosion rapid until 1970. Protection works slowed erosion.</td>
</tr>
<tr>
<td>1.0 – 1.55 (Site 1)</td>
<td>L</td>
<td>-10 to -14m</td>
<td>Erosion consistent through time.</td>
</tr>
<tr>
<td>1.1 – 1.5</td>
<td>R</td>
<td>0 to -17m</td>
<td>Inside of bend. Dredging has altered erosion rates.</td>
</tr>
<tr>
<td>1.55 – 2.5</td>
<td>L</td>
<td>+5 to -5m</td>
<td>Relatively stable except for upstream end. Low tide bench lined with reeds.</td>
</tr>
<tr>
<td>1.6 – 2.4</td>
<td>R</td>
<td>-7 to -18m</td>
<td>Erosion rapid from 1930 to 1949. Protection works slowed erosion.</td>
</tr>
</tbody>
</table>

**Notes:**
1. See Figures 2.3 & 3.6.
2. As seen looking downstream.
3. – erosion + accretion.
Based on linear regression analysis of the results from past riverbank movements at the two sites (Figure 3.9) it is recommended that a projected erosion rate of 0.25 m/year for Site 1 and 0.15 m/year for Site 2 be used for future bank movements. In addition to this figure a safety margin would need to be added to allow for a variance in circumstances in the future. The magnitude of this safety factor is a planning decision to be made by Bankstown City Council, but may be in range of 15 to 20 m for a 100 year planning period corresponding to a total river bank movement of 35 to 45 m.

A total of 4 bank protection options have been considered (see Figure 4.2). They comprise the 3 options discussed with Bankstown Council and Freehill, Hollingdale & Page, namely placed rock, sheet piling and reno-mattresses, and an additional option involving the placement of grout-filled double sided nylon mattresses. Alternative strategies for treatment of the foreshore above low water level have been considered for each option and have been termed “hard” and “soft” finish (see Figure 4.3).

A summary of estimated construction costs for each option is shown in Table 2. The cost estimates are base construction costs only and do not include allowances for contingencies, design fees and supervision costs. Of the 4 options considered for bank protection below low water, Option “A” and “D” are by far the most attractive based on cost.

Option “A”, the placed rock option, is recommended as the preferred protection measure below low water on the basis of it being a proven technique, relative simplicity of construction, competitive cost and ease of maintenance. It is however also recommended that Option “D”, the grout-filled nylon mattress, be considered further at the detailed design stage.

Either a “hard” or “soft” finish extending Option “A” and “D” above low water is acceptable on technical grounds, provided the “soft” finish is adequately maintained. However, from the viewpoint of environmental acceptability a “soft” vegetative approach would be preferable.
TABLE 2
SUMMARY OF ESTIMATED CONSTRUCTION COSTS

<table>
<thead>
<tr>
<th>OPTION</th>
<th>ESTIMATED CONSTRUCTION COSTS ($A MILLION)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Cost</td>
<td>Total Cost</td>
</tr>
<tr>
<td></td>
<td>&quot;Fully Hard&quot;</td>
<td>&quot;Soft Finished&quot;</td>
</tr>
<tr>
<td>A (Placed Rock)</td>
<td>0.99</td>
<td>0.97</td>
</tr>
<tr>
<td>B (Sheet Piling)</td>
<td>2.36</td>
<td>2.34</td>
</tr>
<tr>
<td>C (Reno-mattress)</td>
<td>1.85</td>
<td>1.83</td>
</tr>
<tr>
<td>D (Grout-filled nylon mattress)</td>
<td>0.90</td>
<td>0.88</td>
</tr>
</tbody>
</table>

Notes:
(a) "Hard" finish assumes 230mm reno-mattresses laid on geotextile fabric to a trimmed slope 1(V):1.5(H), extending from low water to top of existing bank (with objective of minimising cut into existing foreshore).

(b) "Soft" finish assumes inter-tidal beach at slope 1(V):7(H) then battering up to top of bank at a steeper slope but as gentle as possible without encroaching more than 10m beyond the existing top bank.

(c) Costs are based on a combined bank length for Site 1 & 2 of 700m.

(d) Costs do not include an allowance for contingencies, design fees or supervision costs.
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4.1 ESTIMATED CONSTRUCTION COSTS
1. INTRODUCTION

The Georges River (Figure 1.1) is located in the southern part of metropolitan Sydney. The upper reaches are in a bushland catchment while the middle and lower river sections pass through heavily urbanised and industrialised areas. The study area consists of 2.5 km of river foreshore land sited approximately 25 km from the river's mouth at Botany Bay. The eastern portion of this foreshore land is part of the East Hills Golf Course redevelopment site. Two reaches were identified as undergoing bank erosion adjacent to the site and are referred to as Site 1 and 2 respectively, as shown on Figure 1.2.

Bank erosion along the Georges River has been identified in previous studies since the early 1960's. There are a number of potential causes of bank erosion in the vicinity of the study area. These include:

- natural river migration and other bank erosion processes,
- large scale sand extraction from the river adjacent to the site and in the reaches upstream and downstream from it,
- erosion resulting from boat wash,
- wind wave erosion,
- the Chipping Norton Lakes Scheme development which has affected the hydraulic behaviour of the upper tidal portion of the Georges River,
- changes in flood behaviour as a result of increased urbanisation of the catchment, and
- widespread use of piecemeal foreshore protection measures which may have exacerbated bank erosion further downstream.

The geomorphology of the lower Georges River is dominated by two major rock units, Hawkesbury Sandstone and the Wianamatta group of shales. The severity of erosion can be directly related to the influence of these underlying formations. Erosion of the Hawkesbury Sandstone is less pronounced with the river direction primarily controlled by the joint direction of rock. The Wianamatta group of shales is more easily eroded and forms the alluvial floodplain in which the study area is located.

An examination of 1930 aerial photographs shows remnant features where the river has migrated hundreds of metres in the last thousand or so years. Fluvial geomorphological features such as scroll bars and abandoned meander patterns support this theory. As the river has tended to migrate across the floodplain in the past, so it will in the future if not arrested by structural bank protection measures.
This report sets out in detail the historical rates of bank erosion and examines the processes that may be causing the erosion. An assessment of likely long term trends in bank erosion is made with possible remedial options offered to combat the problem.
2. HYDROSURVEY ANALYSIS

The main purpose of the hydrosurvey analysis was to give an indication as to the changes that have occurred in the cross section shape and depth over the period 1926 to 1984. They were not used for changes in river alignment as the photogrammetry was far superior in terms of accuracy.

The following aspects were examined as part of the hydrosurvey analysis:

- Depth of thalweg (line of greatest depth)
- Comparison of cross section shape and depth
- Changes in shoaling patterns.

2.1 DESCRIPTION OF SURVEYS

A preliminary search was made to obtain historical hydrosurveys of the Georges River, concentrating on the study area. Various plans were obtained with the more useful chosen for analysis. The surveys used in this study are listed in Table 2.1.

<table>
<thead>
<tr>
<th>DATE OF SURVEY</th>
<th>DRAWING NO.</th>
<th>COVERAGE</th>
<th>SURVEYOR</th>
<th>TYPE</th>
<th>APPROX. SCALE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feb 1926</td>
<td>* 22058</td>
<td>Liverpool to East Hills</td>
<td>Unknown</td>
<td>Soundings</td>
<td>440 ft to an inch</td>
</tr>
<tr>
<td></td>
<td>* 21114</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Nov 1959</td>
<td>59/191</td>
<td>Botany Bay to Liverpool</td>
<td>PWD</td>
<td>Soundings and cross sections</td>
<td>800 ft to an inch</td>
</tr>
<tr>
<td>Sep 1976</td>
<td>Plan Cat No.</td>
<td>Liverpool Weir to East</td>
<td>PWD</td>
<td>Overlay cross sections</td>
<td>1 : 2000</td>
</tr>
<tr>
<td>Apr 1977</td>
<td>8759</td>
<td>Hills</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>May 1978</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oct 1984</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Refers PWD microfilm number
A brief description of the surveys that were used in the study follows:

**February 1926 : Hydrosurvey (Figure 2.1)**
These maps cover the Georges River from Liverpool to East Hills. The survey consists of depth soundings along roughly the thalweg of the main channel. The plan is useful for thalweg comparison purposes. A 1938 hydrosurvey shows similar depths to those obtained but was not analysed due to discrepancies in position reproduction. The 1926 plan was the only survey analysed prior to any dredging operations in the East Hills/Milperra area.

**November 1959 : Hydrosurvey (Figure 2.2)**
This was a major survey carried out by the PWD and covered the area from Botany Bay to Liverpool. The survey consisted of depth soundings, cross sections and a tidal plane analysis along the river. Dredging operations had been occurring in the study area some 10 years before this survey was undertaken.

**September 1976, April 1977, May 1978 and October 1984 Overlay Cross Sections (Figure 2.6 A to G)**
This series of surveys by the PWD is the most recent available and covers the river from East Hills to Liverpool Weir. It includes a comparison of cross sections for the above four survey dates. The 1984 survey was used as a base from which to compare the other surveys. The overlay comparison surveys were initiated as part of the data collection activities monitoring any effects of Chipping Norton Lakes Scheme on the hydraulic behaviour of the river.

2.2 **ACCURACY OF SURVEY DATA**

When comparing the depth soundings of one survey to another it is necessary to consider certain factors that may influence the comparison. The main factors include the following:

**Position Replication**
This is the main factor affecting cross section and thalweg comparisons especially in the 1926 and 1959 surveys. Due to variability in river bathymetry over short lengths, mismatch in position location of the compared sections may record changes in bathymetry along the river length instead of only across the section. Probable error in reproduction in plan view is of the order of 5m.

**Datum Uncertainties**
Often older surveys are imprecise as to the height of the adopted water level the depth soundings are referred to. It is necessary to convert the depths on the survey to a common datum (ie. Standard Datum)
For the 1926 survey the datum used was Low Water Ordinary Spring Tides (LWOST). It has been assumed that a stepped datum similar to the 1959 tidal planes was adopted for the survey from the ocean upstream to the study area. The 1959 value of Mean Low Water Springs (MLWS) has been assumed indicative of the datum on this plan, i.e. the datum for 1926 survey is taken as –0.5m (Standard Datum). No conversion was necessary for the 1959, 1976, 1977, 1978 and 1984 as all soundings were reduced to Standard Datum.

Other Factors

Other factors such as changed techniques in obtaining soundings and the presence of mobile bedforms can introduce anomalies in height when comparing different surveys.

Long term trends in bathymetry may be altered as a result of short term fluctuations due to flood scour. A flood history at Liverpool Weir is given at Figure 2.4. Due to relatively similar antecedent flood conditions, changes in bathymetry are unlikely to be attributable to this factor for the 1926, 1959, 1977 and 1984 surveys. The two exceptions to this are the September 1976 and May 1978 surveys, which were preceded by the June/July 1975 floods and the March 1978 flood respectively. Flood scour is especially severe in the May 1978 survey which was undertaken about two months after the occurrence of a moderate flood on the Georges River.

It is estimated that taking into account datum information on plans and the conversion of soundings to Standard Datum, the maximum error was limited to 0.3m.

2.3 TRENDS IN THALWEG DEPTHS

Longitudinal profiles of maximum channel depths (i.e. thalweg) derived from the 1926, 1959 and 1984 hydrosurveys of the Georges River are shown as Figure 2.5. The graph shows the thalweg has progressively deepened since it was first surveyed in 1926. CS21 to CS41 denotes the cross section numbers used in this study.

Extensive dredging of the main river channel in the study area in the 1950's and 1960's was the primary cause of channel deepening.

Between CS21 and CS26 the depth was relatively consistent for the period 1926 and 1984, except for a localised shoal between CS23 and CS24 which was removed sometime in the 1950's. The channel has deepened approximately 1m between CS21 and CS23 probably as a result of the thalweg meandering hard up against the left bank (refer Figure 2.6 Sheet A). According to Warner and Pickup (1973) there has never been any dredging in this segment of the river, i.e. adjacent to the East Hills Golf Clubhouse.
Between CS26 to CS30 there was an average deepening of the main channel of around 2.5m from 1926 to 1959. This was a result of extensive dredging in the 1950's. Figure 3.2 shows a 1951 aerial photograph with a dredge working in this river segment.

From 1959 to 1984 the area CS26 to CS28 was relatively stable, with the intensity of dredging operations reduced. For the period 1959 to 1984 between CS28 and CS31 there was an increase of around 3m in average depth, with a local maxima of 4m at CS29. It appears that dredging operations were concentrated in this area in the 1960's, i.e. opposite Site 1.

For the reach CS32 to CS40 between 1926 and 1959 there was an average increase in depth of 3.0m, with localised maxima in the vicinity of CS35 and CS36 of around 6m. The majority of this dredging was carried out in the 1950's. This reach was relatively stable between 1959 and 1984 except for a localised area between CS37 and CS39.

In overview from 1926 to 1984, the river depth in the study area has undergone massive changes as a result of extensive dredging carried out in the 1950's and 1960's. Depths have tripled at some locations. Whereas in 1926 the river section had an average depth of around 3 to 4m and a maximum of 6m, today it averages around 6 to 7m with local holes up to a maximum of over 10m.

The location of the 1984 thalweg is shown on Figure 2.3. The plan view of the channel thalwegs in 1926 and 1959 could not be accurately located. As an overview the thalweg shows a progressive meandering trend with the more intense areas of bank erosion distributed in an alternating pattern from left bank to right along the river section.

2.4 CROSS SECTION COMPARISON

Twenty one cross section locations were selected as shown in Figure 2.3 and cross sections were prepared (see Figure 2.6 Sheet A to G). The information provided by the cross sections is only reliable for the overlay cross sections from 1976 to 1984 as the 1959 sections rarely matched up in horizontal position with the overlay sections. Thus, the 1959 survey sections are only shown for general section shape and not as a straight comparison with the overlay sections. Also, the short time period (i.e. 12 years) that the overlay sections extend over, makes it impracticable to ascertain any long term changes in river channel morphology.

At Site 2 in the vicinity of the Clubhouse the main river channel is hard up against the left bank, with underwater bank slope of up to 1(V):1.5(H) extending down to a depth of 7m (refer to CS22 and CS23). Warner and Pickup (1973) have stated that the area adjacent to the Clubhouse has never been dredged and that a comparison...
of cross sections in this area for 1959 and 1973 show this area to be relatively stable. Generally, the 1978 and 1984 surveys tend to be on average 0.5m deeper than the 1976 surveys.

The main channel crosses from left bank to right bank between CS24 and CS25 and hugs the outside of the bend until just before CS27.

The river channel then moves back across to the left bank at around CS30. CS27 to CS31 are shown to be relatively symmetrical sections probably as a result of the extensive dredging in the area. Comparing the thalweg depths with cross section shape suggests that the cross sectional area has up to tripled in the vicinity of CS29 as a result of the dredging in the 1950's and 1960's. The localised hole at CS29 has a cross sectional area of around 660 m$^2$ while the average for this segment of river is between 300 to 400 m$^2$. At CS29 note the scoured bed condition of the post flood 1978 survey, and the relative infilling by tidal flows to the 1984 section. CS30 shows a 1959 survey at the same location as the overlays, indicating that dredging in the 1960's has increased the average depth by around 2m and made the section more symmetrical.

The sections CS31 to CS41 have been relatively stable over the period 1976 to 1984, however, the 1978 survey tends to show a post flood scoured condition. The 1959 surveyed cross section on CS34 and CS35 tends to suggest that dredging was being carried out at the time of the survey, due to the irregular shape of the section.
3. PHOTOGRAMMETRIC ANALYSIS

The detailed photogrammetric analysis was undertaken from the earliest available good quality aerial photographs (1930) to recent (1988). From this detailed analysis contemporary rates of riverbank erosion or accretion were determined.

3.1 METHODOLOGY

Selection of Photography

All available aerial photography of the Georges River was assembled and assessed as to its suitability for use in the study. A list of the available photography is given in Table 3.1. Following a preliminary examination of the complete photographic record, combined with an assessment of photographic quality, accuracy of horizontal control, and the temporal spacing of flights (especially in relation to the flood history of the Georges River), five dates of photography were chosen for detailed photogrammetric analysis, as shown below

<table>
<thead>
<tr>
<th>DATE</th>
<th>TIME PERIOD BETWEEN PHOTOGRAPHS</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 January 1930</td>
<td>19 years, 2 months</td>
</tr>
<tr>
<td>31 March 1949</td>
<td>21 years, 4 months</td>
</tr>
<tr>
<td>July 1970</td>
<td>7 years, 9 months</td>
</tr>
<tr>
<td>13 May 1978</td>
<td>18 years 3 months</td>
</tr>
<tr>
<td>4 November 1988</td>
<td>10 years, 6 months</td>
</tr>
</tbody>
</table>

It was intended to analyse the June 1961 photography, however, this was not undertaken due to distortion in the film reproduction. Some of the aerial photographs are reproduced as Figures 3.1 to 3.4.
<table>
<thead>
<tr>
<th>DATE</th>
<th>TITLE</th>
<th>FILM NO.</th>
<th>CAMERA</th>
<th>HOOD LENGTH</th>
<th>SOURCE</th>
<th>FLYING HEIGHT</th>
<th>SCALE</th>
<th>COVERAGE</th>
<th>QUALITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-11-88</td>
<td>CUMBERLAND *</td>
<td>QAS 2608C</td>
<td>WILD RC10</td>
<td>153 mm</td>
<td>QASCO</td>
<td>8,500 ft</td>
<td>1:17,000</td>
<td>OK</td>
<td>REASONABLE</td>
</tr>
<tr>
<td>2-8-86</td>
<td>SYDNEY COLOUR</td>
<td>NSW 3527</td>
<td>WILD RC10</td>
<td>153 mm</td>
<td>LANDS DEPT</td>
<td>8,000 ft</td>
<td>1:16,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>13-2-84</td>
<td>SYDNEY ISG REVISION</td>
<td>NSW 3385</td>
<td>WILD RC10</td>
<td>152 mm</td>
<td>LANDS DEPT</td>
<td>8,000 ft</td>
<td>1:16,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>9-8-82</td>
<td>SYDNEY</td>
<td>NSW 3241</td>
<td>WILD RC10</td>
<td>151 mm</td>
<td>LANDS DEPT</td>
<td>8,000 ft</td>
<td>1:16,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>13-5-78</td>
<td>CUMBERLAND *</td>
<td>NSW 2713</td>
<td>WILD RC10</td>
<td>151 mm</td>
<td>LANDS DEPT</td>
<td>8,000 ft</td>
<td>1:16,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>24-11-76</td>
<td>CHIPPING NORTON TO GEORGES RIVER</td>
<td>NSW 3017</td>
<td>WILD RC10</td>
<td>152 mm</td>
<td>LANDS DEPT</td>
<td>5,000 ft</td>
<td>1:10,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>2-4-75</td>
<td>CUMBERLAND</td>
<td>C-85</td>
<td>WILD RC10</td>
<td>152 mm</td>
<td>QASCO</td>
<td>8,000 ft</td>
<td>1:16,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>16-11-73</td>
<td>SYDNEY ISG</td>
<td>NSW 2167</td>
<td>WILD RC10</td>
<td>151 mm</td>
<td>LANDS DEPT</td>
<td>8,000 ft</td>
<td>1:16,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>7-7-70</td>
<td>CUMBERLAND *</td>
<td>NSW 1908</td>
<td>WILD RC10</td>
<td>152 mm</td>
<td>LANDS DEPT</td>
<td>7,500 ft</td>
<td>1:15,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>25-6-61</td>
<td>CUMBERLAND</td>
<td>NSW 1046</td>
<td>WILD RC5</td>
<td>210 mm</td>
<td>LANDS DEPT</td>
<td>9,200 ft</td>
<td>1:14,000</td>
<td>OK</td>
<td>DISTORTED</td>
</tr>
<tr>
<td>MAY '51</td>
<td>CUMBERLAND</td>
<td>NSW 466</td>
<td>EAGLE 9</td>
<td>305 mm</td>
<td>LANDS DEPT</td>
<td>12,200 ft</td>
<td>1:12,200</td>
<td>50%</td>
<td>GOOD</td>
</tr>
<tr>
<td>31-3-49</td>
<td>LIVERPOOL *</td>
<td>SVY 549</td>
<td>FAIRCHILD K17</td>
<td>154 mm</td>
<td>AUSLIG</td>
<td>16,000 ft</td>
<td>1:32,000</td>
<td>OK</td>
<td>GOOD</td>
</tr>
<tr>
<td>10-2-30</td>
<td>SYDNEY *</td>
<td>MAP 3429</td>
<td>F8</td>
<td>178 mm</td>
<td>AUSLIG</td>
<td>12,500 ft</td>
<td>1:21,000</td>
<td>OK</td>
<td>POOR</td>
</tr>
</tbody>
</table>

* Denotes photography analysed for this study
The 1930 to 1988 photography provides the largest time span to assess trends, while the infill photography (1949, 1970, 1978) was selected to be approximately equally spaced at two decade intervals or to reflect flood incidence. Comparison with the flood history (Figure 2.4) shows the period 1930 to 1949 had only three minor to moderate floods, 1949 to 1970 had a high flood incidence with the highest flood this century occurring in November 1956, 1970 to 1978 had three minor to moderate floods late in the time period while 1978 to 1988 experienced two moderate to serious floods late in the photographic time period. It is then possible to compare variability in erosion rates determined from these periods to ascertain whether flood events are a significant cause of bank erosion.

**Method of Analysis**

Photogrammetric digitising of the features was carried out on a Wild AC1 Analytical Stereoplotter used to analyse aerial or terrestrial photography. The resolution of this instrument as specified by the manufacturers is one micron, at the scale of photography. Each stereo pair of aerial photographs was set up in the AC1 and using the reference mark in the optical system the features were digitised.

The data captured included bank scarps, cultural features (eg buildings and roads), riverbank vegetation, levees and shoal features. The data was then plotted out on a Wild TA2 Automatic Plotting Table at a scale of 1:2000 for each year of photography. The Central Mapping Authority ISG co-ordinate grid system was used. The photogrammetric sheet layout is shown at Figure 3.5. Changes in scarp movement over time are established by overlaying 2 different sheets, carefully aligning the grid and scaling off the relative distances.

**Photogrammetric Accuracy**

The accuracy of the photogrammetric analysis depends on the following factors:

- quality of aerial photography,
- quality of ground control,
- instrument used and its resolution,
- method of aerial triangulation, and
- accuracy of the operator and the plotting procedure.
The accuracy of vertical and horizontal measurements obtained for each year of photography is shown in Table 3.2. Note that the figures quoted are upper limits and that majority of data observed were within these ranges.

### Table 3.2
**Summary of Photogrammetric Accuracy**

<table>
<thead>
<tr>
<th>Date of Photography</th>
<th>Horizontal Accuracy</th>
<th>Vertical Accuracy</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>1988</td>
<td>±0.3m</td>
<td>±0.3m</td>
<td>Fuzzy Imagery</td>
</tr>
<tr>
<td>1978</td>
<td>±0.2m</td>
<td>±0.2m</td>
<td>OK</td>
</tr>
<tr>
<td>1970</td>
<td>±0.2m</td>
<td>±0.2m</td>
<td>OK</td>
</tr>
<tr>
<td>1949</td>
<td>±0.8m</td>
<td>±0.6m</td>
<td>High level photography</td>
</tr>
<tr>
<td>1930</td>
<td>±1.5m</td>
<td>±0.7m</td>
<td>Distorted uncalibrated photography</td>
</tr>
</tbody>
</table>

### 3.2 River Scarp Data Analysis

**River Scarp Movements**

Figure 3.6 Sheets A to D show the scarps superimposed from each year of photography analysed, at a scale of 1:2000. The layout of the photogrammetric Sheets is shown on Figure 3.5. The relative movement of the scarp was measured at 20m intervals along the centreline of the river, for both the right and left banks (as viewed looking downstream). From this data the rate of scarp movement was obtained over the period of analysis and the results are shown as Figure 3.7 and 3.8.

In the photogrammetric analysis of bank scarps, erosion is more readily identified than accretion. This is because erosion is more readily defined as a sharp retreat of the bank, while accretion tends to occur on shoals rather than as a progradation of the river bank. In certain areas, where vegetation has obscured the scarp, the location of the scarp is shown as a dotted line with the accuracy reduced. In several areas there was no well defined scarp for many of the years, so that no line was plotted and thus no data points were obtained. These areas are labelled, as “no observations” areas on Figure 3.7.
Regression Analysis

A linear regression analysis was also carried out on the data at selected profiles for Site 1 and Site 2. The location of these profiles (8 profiles at Site 1, 6 profiles at Site 2) are shown on Figure 3.6 Sheet A to C with the individual linear regression plots shown on Figure 3.10 and 3.11 for Site 1 and 2 respectively. The line plotted is the result of a regression analysis using all dates of photography. The slope of the line represents the historical trend in scarp movement, e.g. a slope of 0.2 represents an average erosion rate of 0.2 m/year. The "R SQR" number quoted for each profile is a statistical indicator used for quality of fit. If R SQR is equal to one then the data used is a perfect fit to the calculated line. As the scatter of data points from the line of best fit increases, the value of R SQR decreases towards zero.

The results of the linear regression analysis is shown in graph form at Figure 3.9. The rate of scarp movement using linear regression is plotted against chainage along the river for Site 1 and 2.

3.3 DISCUSSION OF RESULTS

3.3.1 Scarp Movement Through Time

km 0 — km 0.45 (Left Bank)

In the vicinity of Site 1 the left bank has eroded on average 5–10m over the period 1930 to 1984. The riverbank at this location is fronted by a medium covering of trees which has not changed to any great extent over the period of analysis. The channel invert is hard up against the toe of the bank.

Erosion through time has been somewhat erratic as shown by the variable rates on Figure 3.8. On average, the erosion rate for 1930–49 was the highest (0.25 m/year) over the period of analysis. This period was noted for its low flood incidence and absence of any dredging operations or wave attack from boat wash. The rate drops off to about zero from 1949–70, then changes to an accretion rate of 0.15 m/year for the period 1970–78. The variability in rates during the 1949 to 1978 time period was probably due to intermittent rock protection and minor reclamation works which have masked the natural erosion rate. Plate 4 shows the remains of past protection works such as concrete slabs scattered on the low tide bench at this location. This is especially evident for the period 1970–78. For 1978–88 the rate has once again increased to an average of about 0.20 m/year, similar to 1930–49. As an overview, the reclamation works in this area have altered the erosion rate through time, with the strong possibility that the inherent rate is a lot higher than the photogrammetric analysis reveals.
km 0 – km 0.4 (Right Bank)

The bank has been relatively consistent over time, with the low tide bench at this location heavily covered by mangroves. The mangroves have made identifying the scarp location difficult.

km 0.45 – km 1.0 (Left Bank)

This bank segment is at the inside of a bend and appears relatively stable or slightly accreted over time. The thick mangrove covering again makes scarp identification difficult. From km0.4 to km0.58 a land filling operation around 1970 reclaimed an area that was originally a mangrove swamp area.

km 0.45 – km 1.0 (Right Bank)

The thalweg crosses from left to right bank at approximately km0.45. There has been appreciable erosion at the outside of this river bend of between 10 and 15m over the period 1930–88. Erosion through time has varied with the highest rates occurring in the earlier time period between 1930–49, with a maximum of 0.7 m/year between km0.75 and km0.85. The rate has gradually reduced since then, with the latest period (1978–88) being relatively stable. The reduction in rate is mainly due to the random dumping of concrete slabs and bricks along this section of the bank over the last 10 years. It appears that very heavy dumping of reject house bricks along the foreshore has armoured the underwater face of the bank and arrested long term bank erosion for the present.

km 1.0 – km 1.55 (Left Bank)

This section of riverbank is referred to as Site 1 and is the most susceptible to bank erosion processes affecting the development site. From 1930 to 1988 there was erosion of the scarp in this area of up to 14m. The erosion occurred on the outside of a bend in the river. The bank height at this location ranges from 1 to 1.5m and is characterised by a general lack of vegetation fronting the bank. An examination of aerial photography over the last 60 years shows that there has been little change in the vegetation cover over time. The scarp is fronted by a low tide bench with its maximum width at the upstream end, then reducing in a downstream direction. The soil is mainly silty in nature being held together at the top of the bank by a thick covering of grass roots, which is easily undercut. A recent site inspection revealed appreciable bank slumping along this stretch of river.

The erosion rate has been relatively consistent at between 0.2 to 0.25 m/year over the period of analysis. The consistent rate is indicative of a river that is experiencing bank erosion primarily as a result of natural river migration. Thus the river has still eroded at a relatively consistent rate even though
other factors such as flood incidence, boat wash and dredging operations have varied over the period of analysis.

km 1.1 – km 1.5 (Right Bank)

This length of riverbank is opposite Site 1 and is located on the inside of a bend. The river is fronted by a medium covering of mangroves along the low tide bench. Erosion of the scarp has occurred over the last 60 years, with a maximum bank loss of around 17m. Erosion at this location is in contrast to the normal trend for a natural meandering river, where erosion occurs at the outside of a bend and accretion at the inside. This exception to the rule can be explained by dredging operations that occurred between 1950 and 1970, during which time this bank was partially removed. Figure 3.3 shows a 1961 aerial photograph with two dredges removing the low tide bench at this section of the river.

km 1.55 – km 2.5 (Left Bank)

The thalweg swings from the left bank to the right at between CS32 and CS33. This section of riverbank is fronted by a low tide bench of between 10 to 15m and is generally covered by phragmite reeds. The riverbank has been relatively stable over the last 60 years with an average erosion rate fluctuating between 0.05–0.10 m/year for this period. There has been a tendency for erosion rates to be higher in the last 20 years than from 1930 to 1949. This may be as a result of dredging carried out between 1950 to 1970 which may have altered the natural bank erosion processes.

km 1.6 – km 2.4 (Right Bank)

The thalweg crosses to the right bank of the river at approximately chainage km 1.65. There has been significant erosion up to a maximum of 18m over the period of analysis. Erosion through time has varied with the highest rate occurring in 1930 to 1940, when the maximum rate exceeded 0.5 m/year between km 1.9 and km 2.2. The rate reduced to 0.2 to 0.3 m/year for the period 1940 to 1970. For 1970 to 1978 the apparent trend is one of accretion with the latest period (1978 to 1988) showing a relatively stable scarp. The variation in rates is mainly attributable to the heavy dumping of concrete blocks, bricks, tiles and building refuse along this section of riverbank over the last 20 years. It appears that this dumping has slowed the erosion rate but has not completely stopped it as evidenced by the remnant concrete blocks on the low tide bench fronting this portion of riverbank.
3.3.2 Regression Analysis

The linear regression plots for Sites 1 and 2 are shown on Figures 3.10 and 3.11 respectively. From these results the rate of linear regression is plotted against chanmage along the river as shown in Figure 3.9.

For Site 1 the rate of recession over time has been remarkably consistent with the statistical indicator R squared generally in the range of 0.95 to 1.0. This indicates that the line of best fit gives a good correlation to the data points plotted, with minimal scatter. Figure 3.9 shows that the average scarp movement over time for the river bend between km1.06 and km1.40 is consistently between 0.2 and 0.25 m/year. Figure 3.6 Sheet B shows this section of river to have undergone the most erosion. This erosion is located at the apex of a bend in the river with a slight bias to the downstream end of the bend. The consistent rate of scarp movement through time at this location is indicative of a river undergoing natural river migration, with the rate relatively unchanged during different periods of flood incidence, dredging operations and boat traffic.

For Site 2 the rate of scarp movement has been more erratic and not as high as for Site 1. Figure 3.9 for this site shows the average scarp movement to be generally in the range of 0.1 to 0.15 m/year. This lesser rate compared to Site 1 is not surprising considering that the curvature of this bend is much less than at the other site. The variability in rate over time is mainly due to reclamation works carried out along this section of foreshore land. These works have included the random dumping of concrete slabs, bricks and other building refuse, and have not been successful in arresting the erosion of the bank. Uncertainty regarding the extent to which these reclamation works have affected the natural erosion rate necessitates caution in the use of historical rates for the prediction of future erosion trends.

3.4 CONCLUSIONS AND RECOMMENDATIONS

The Georges River in the vicinity of the study area has shown a general trend over time of erosion/accretion as the river has migrated across the alluvial floodplain. This classical meander pattern, with erosion at the outside of a bend and accretion at the inside of the bend is predominant on most river bends in the study area. There are some exceptions to this rule, but they can be mainly attributed to man's interference with the natural river processes e.g. extensive dredging operations. Taking the river length bounded by the study area, erosion is clearly the dominant bank trend.

Approximately 1.5 km of riverbanks have been affected to various degrees by piecemeal foreshore protection works, with the majority of these works located on the right bank of the river opposite the redevelopment site. These works consist mainly of concrete slabs, bricks, tiles and other building refuse. The effect that these
protection works have had on arresting erosion has varied from one location to the next depending on the extent of protection for the toe of the eroding riverbank. It appears that the very heavy dumping of reject house bricks between km0 45 and km1 0 on the right bank has armoured the underwater face of the bank and arrested long-term bank erosion for the present. At other locations the random dumping of debris has had little effect on the arresting bank erosion.

Apart from cases where the bank erosion rate has been altered by reclamation and/or foreshore protection works, the rate has been generally consistent over the period of analysis, i.e. the last 60 years. This tends to confirm the view that the main cause of bank erosion is due to natural river migration, with other factors such as erosion from boat wash and dredging not present over the first 20 years of analysis (1930–1950). General experience suggests that flooding is a contributing mechanism to bank erosion, however the results suggest that different periods of high and low flood incidence have had little effect on the rate of bank erosion.

The majority of the eroding riverbanks have a low tide bench fronting the near vertical scarp. The underwater bank generally slopes very abruptly away from this low tide bench. This suggests that an additional process is operating above water level. The width of the low tide bench tends to be narrowest at the apex of an eroding river bend.

Two reaches fronting the development site have been identified as experiencing bank erosion and are referred to as Sites 1 and 2 (Figure 1.2). The upstream Site 1 shows a relatively constant recession over the last 60 years of 0.2–0.25 m/year. Downstream at Site 2, adjacent to the Golf Course Clubhouse, the results show a reduced rate and increased variability over time compared with Site 1. Rates have fluctuated over time but are generally in the range of 0.1–0.15 m/year for the period 1930 to 1988. Fluctuations may have largely been due to random bank protection works and minor land reclamation works adjacent to the riverbank.

Based on a linear regression analysis of the results obtained from past riverbank movements at the respective sites, it is recommended that a projected erosion rate of 0.25 m/year for Site 1 and 0.15 m/year for Site 2 be used for future planning. In addition to this figure a safety margin would need to be added to allow for a variation in circumstances in the future e.g. increased urbanisation, greenhouse effect. The magnitude of this safety margin is a planning decision to be made by Bankstown City Council, but may be in the range of 15 to 20 m for a 100 year planning period. Project planning would therefore be based on a cumulative riverbank movement in the order of 35 to 45 m for a 100 year design life.
4. FORESHORE PROTECTION OPTIONS

4.1 BASIS OF APPROACH

As indicated in Chapter 3 above, the predicted likely bank movements at Sites 1 and 2 (which together comprise a total bank length of 700m, see Figure 4.1) are 25 and 15m respectively based on a 100 year planning period. It was also suggested that an additional 20m be allowed as a safety factor to allow for a variance in future hydraulic and hydrologic conditions. Project planning is therefore based on a total river bank movement in the order of 35 to 45m within the 100 year design life.

The low cost response to the predicted bank migration is to accept the progressive movement of the bank and plan development accordingly. This is called the “non-structural” approach as no investment in foreshore protection structures is involved.

Discussions with Freehill, Hollingdale and Page (representing the developer) and Bankstown Council however have indicated that a bank movement of 35 to 45m would impose a severe constraint on the golfcourse layout. It was therefore requested that “structural” options be considered to control bank migration.

Assessment of structural options has been undertaken based on the following considerations:

- “Partial” bank protection extending from design toe depth up to approximate low water level.
- Alternative strategies for treatment of the foreshore above low water level.

A total of 4 “partial” bank protection options has been considered (see Figure 4.2). These comprise the 3 options discussed with Bankstown Council and Freehill’s, namely placed rock, sheet piling and reno-mattresses, and an additional option involving placement of grout–filled double sided nylon mattresses. It was decided to include this additional option in the analysis following discussions with contractors experienced in river protection works, who suggested the grout–filled nylon mattress as a more viable option than reno-mattresses or sheet piling.
Alternative strategies for treatment of the foreshore above low water level (see Figure 4.3) comprise:

- a “soft” finish comprising native regeneration of a battered zone extending from low water up to existing top of bank level (approx 0.5 to 1m above high water), and
- a “fully hard” finish involving placement of reno-mattresses from low water up to the top of the existing bank

A description of the technical features of each option is given in Sections 4.2 and 4.3 below. A summary of the estimated construction costs for each option is presented in Table 4.1.

**TABLE 4.1**

**ESTIMATED CONSTRUCTION COSTS**

<table>
<thead>
<tr>
<th>OPTION</th>
<th>ESTIMATED CONSTRUCTION COSTS (in $A millions)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) Placed Rock</td>
<td>(2)</td>
</tr>
<tr>
<td>A</td>
<td>0.81</td>
</tr>
<tr>
<td>B (Sheet Piling)</td>
<td>2.18</td>
</tr>
<tr>
<td>C (Reno-mattress)</td>
<td>1.67</td>
</tr>
<tr>
<td>D (Grout-filled nylon mattress)</td>
<td>0.72</td>
</tr>
</tbody>
</table>

**Notes:**

(a) "Hard" finish assumes 230mm reno-mattresses laid on geotextile fabric to a trimmed slope 1(V):1.5(H), extending from low water to top of existing bank (with objective of minimising cut into existing foreshore).

(b) "Soft" finish assumes inter-tidal beach at slope 1(V):7(H) then battering up to top of bank at a steeper slope but as gentle as possible without encroaching more than 10m beyond the existing top bank.

(c) Costs are based on a combined bank length for Site 1 & 2 of 700m.
Cost calculations for each option together with a schedule setting out the basis of cost rates used are included at Appendix A.

It is noted that costing for all options assumes that any material excavated from the river bank is dumped 10m from the existing river bank for eventual re-use for golfcourse development bulk earthworks. Provision has not been made for use of excavated material to fill deep "holes" opposite Site 1, as it is considered doubtful that placement of such fill will be sufficient to permanently raise the river bed at these locations.

4.2 PROTECTION BELOW LOW WATER

4.2.1 Option A: Placed Rock

Option A, as with Options C and D, initially involves excavation and trimming of the existing bank profile below low water to a slope of 1(V) 1.5(H). A geotextile fabric is then to be laid on the trimmed slope and rock placed on top of the fabric to a total layer thickness of 0.75m.

Minimum size of rock protection has been calculated as 15 kg taking into account expected flood velocities, angle of trimmed bank slope and river channel sinuosity [Zanen, 1981]. For relatively dense rock such as basalt this is equivalent to a size of 225mm. Flood velocity used in the calculation was 2.5 m/sec based on recorded flood velocities in the vicinity [PWD, 1987 & 1989].

It is proposed to continue the rock protection down to a level equivalent to the existing thalweg depth. An additional quantity of rock is to be placed at the toe with the intention that the additional rock will tend to roll down the slope and "extend" the protection should the protection be undercut. This is considered important as it is well established that the presence of a revetted river bank may in itself cause development of a deeper bed locally [Zanen, 1981]. Making the design flexible so that the revetment can adapt itself to any changes in the river bed is therefore a key design consideration. The quantity of additional rock included at the toe is 8 m³/m which is theoretically equivalent to extending the protection another 3m in depth.

Geotextile fabric is incorporated into the design to prevent soil particles escaping through the rock layer. This is considered important given the desired long design life (100 years) and the fact that the Chipping Norton Lakes Scheme has increased the natural tidal prism at the golfcourse site and hence has resulted in increased tidal currents.
The advantages of the placed rock option are cost and ease of maintenance. It is the cheapest of the options considered with an estimated construction cost of $810,000. The rockfill can easily be topped up if maintenance/repair are found to be necessary during the design life. If mangroves are established above low water any topping up of rockfill will result in disturbance of the mangroves. Nevertheless this is not considered a major drawback as “maintenance” will only be necessary if mangroves start to be lost through bank erosion. If this is the case it is felt there will be community acceptance of the need to obtain access to stabilize the situation.

Construction of a placed rock revetment will also be comparatively straightforward. The rockfill can be placed using “super-reach” excavators or draglines. A large amount of rock will need to be trucked in, which can cause a negative impact on urban areas through excessive truck movements. In this case, however with the main access route being arterial road Henry Lawson Drive, no significant impact on residential areas is likely.

4.2.2 Option B: Sheet Piling

Option B involves driving sheet piling a short distance from the existing bank to provide bank protection. For purposes of preliminary costing the depth of piling has been taken as depth of existing thalweg plus a 3m safety factor to allow for any future bed deepening and a 50% embedment length for stability. This results in driving depths of 12 to 16m.

Sheet piling is no longer produced in Australia and is imported, mainly from Japan. Mitsubishi Australia advises that delivery time is approximately 6 to 7 months from placement of order. Sections are available in thicknesses ranging from 8 to 16mm. A thickness of 10.5mm was adopted for preliminary costing purposes.

Driving to depths of 12 to 16m will require a crawler crane with a drop hammer (for shallower depths an excavator is sufficient). Driving costs are high.

The principal drawback of the sheet piling option is cost. At an estimated construction cost of $2.2 million, it is nearly 3 times more expensive than the placed rock option.

4.2.3 Option C: Reno-mattresses

Option C, as with the placed rock option, initially involves excavation and trimming of the existing bank profile below low water. A geotextile fabric is then to be laid on the trimmed slope and reno-mattresses of thickness 230mm placed on top of the fabric.
When laid above low water, the PVC-coated wire cages are placed first then filled with rock before being closed and secured. When laid underwater however, it is necessary to initially lay the cages out on dry ground, fill them with rock, close and secure, then move them by crane (or barge with a tilting floor) and place them underwater. This adds considerably to cost. Based on advice from a contractor well experienced in river and foreshore protection works, including reno-mattress work on the Georges River although to much shallower depths, the construction cost for this option is estimated at $167 million. This is over twice as expensive as the placed rock option.

4.2.4 Option D: Grout Filled Nylon Mattress

Option D is the additional option considered suggested by contractors contacted in the course of pricing Options A, B and C.

It also involves excavation and trimming of the existing bank profile below low water. A double-sided nylon mattress is then placed on the trimmed slope and filled with cement grout. This revetment mattress system is a patented product and is marketed in Australia by Foreshore Protection Pty Ltd. It is the policy of Foreshore Protection not to supply the materials to contractors as installation requires specialist skills and the firm feels inadequate installation of its product by inexperienced contractors would harm the company's overall marketing efforts. Supply and installation is therefore carried out by the one organisation.

The Foreshore Protection revetment system provides for the optional incorporation of a collapsible hinge joint at the toe to prevent progression of underscouring. Preliminary cost estimates include therefore allowance in the toe design for provision against deepening of the existing thalweg by a further 3m, as with the Option A, B and C above.

The Foreshore Protection revetment mattress system was recently used at the Garden Island Fleet Base where approximately 9000 m² was laid to depths of 20m and designed to withstand propeller wash from large naval vessels. It would therefore appear suited to the proposed application on the Georges River.

Based on written advice from Foreshore Protection the estimated construction cost of Option D is $810,000, the same as the placed rock option.
Alternative strategies for treatment of the foreshore above low water level have been considered and have been termed “hard” and “soft” finish.

A “soft” or “hard” finish only above low water level, without any additional works below low water, was not considered an effective foreshore protection measure because of undercutting of the works from natural river migration. As such, works above low water were only considered in conjunction with “structural” options below low water.

4.3.1 Hard Finish

The “hard” finish involves use of permanent “hard” materials to stabilize and protect the bank above low water, laid at a sufficiently steep grade to minimize cut into the existing foreshore area. It has been costed on the basis of 230mm thick reno-mattresses laid on geotextile fabric to a trimmed slope of 1(V):1.5(H), extending from mean low water springs to top of existing bank.

The total estimated construction cost of the “hard” finish for the 700m of bank protection works considered is $180,000.

4.3.2 Soft Finish

The “soft” finish involves earthworks and landscaping to create a “soft” appearance characterized by gentle batters and native planting. It has been costed on the basis of forming an inter-tidal beach at a slope of 1(V):7(H) between mean low water springs and mean high water springs, and then battering up to the top of the existing bank at a steeper slope but as gentle as possible without encroaching more than 10m beyond the existing top of bank. Costing provides for marine based planting in the inter-tidal zone of *Aegiceras corniculatum* (river mangrove) and saltmarsh reeds. No provision has been made for terrestrial based planting in the cost estimate.

The total estimated cost of the “soft” finish is $160,000 for the 700m of bank protection works envisaged. This is slightly cheaper than the “hard” finish but undoubtedly annual maintenance costs would be higher.
4.4 CONCLUSIONS AND RECOMMENDATIONS

Of the 4 options considered for bank protection below low water, Options A and D are by far the most attractive options on basis of cost, with estimated construction costs of $0.81 million and $0.72 million respectively. It is noted that the cost estimates are base construction costs only and do not include allowances for physical and/or price contingencies, design fees and supervision costs.

Option A, the placed rock option, is recommended as the preferred protection measure below low water on the basis of it being a proven technique, relative simplicity of construction, competitive cost and ease of maintenance.

It is however also recommended that Option D, the Foreshore Protection grout-filled nylon mattress, be considered further at the detailed design stage. If more detailed cost estimates based on takeoffs from working drawings indicate significant cost savings could be achieved with Option D, then bid documents could be prepared to allow for either construction approach.

Either a “hard” or “soft” finish above low water is acceptable on technical grounds provided the “soft” finish is adequately maintained. However, from the viewpoint of environmental acceptability a “soft” vegetative approach would be preferable.
5. REFERENCES

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2. PUBLIC WORKS DEPARTMENT, NSW

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   "Georges River Flood Data — May 1, 1988". Manly Hydraulics Laboratory, Report No. 553, February 1989

4. SINCLAIR KNIGHT & PARTNERS

5. SINCLAIR KNIGHT & PARTNERS
   "East Hills Golf Course Redevelopment — River Bank Stabilisation along the Georges River". Prepared for Chiyoda Australia Pty Ltd, December 1989

6. WARNER, R. F. & PICKUP, G.

7. WATER RESEARCH LABORATORY

8. ZANEN, A
   "Revetments". International Institute for Hydraulic and Environmental Engineering, Delft, Netherlands, 1981
APPENDIX  A

A1. DEVELOPMENT OF UNIT RATES
A2. QUANTITY CALCULATIONS AND COST ESTIMATE FOR EACH OPTION
### A1. DEVELOPMENT OF UNIT RATES

Prepared 1 June 1990

<table>
<thead>
<tr>
<th>Item No</th>
<th>Item Description</th>
<th>Unit</th>
<th>Rate ($/m³)</th>
<th>Basis</th>
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<td>1</td>
<td>Strip &amp; stockpile topsoil (min 300mm) unsalvageable from existing revetments, then spread &amp; regrade</td>
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<td>$10/m³</td>
<td>spread &amp; grade spoil by machine = $10.75/m³</td>
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</table>
| 2       | Undertake excavation within river to lines & levels shown on drawings  
  (a) for max reach 14m of bank edge  
  (b) for reach between 14m & 25m | m³  |  
  (a) N Scale of Trueway Transport tel advice (049 5159999)  
  o "super-reach" excavator (reach 18.5m) $1200/day  
  o capacity digging underwater, based on Williams R experience at 600 m³/day  
  o est. cost $900 ea way  
  Rate therefore for total qty approx 10,000 m³ = $2.31/m³  
  Seems low use $4/m³ for preliminary costing purposes  
  
  (b) N Scale of Trueway Transport tel advice (049 5159999)  
  o "monster-reach" excavator (reach 33m) $2200/day  
  o assume production rate 60% of above  
  o est. cost $1500 ea way  
  Rate therefore for total qty approx 10,000 m³ = $6.41/m³  
  Use $7/m³ for preliminary costing purposes |
| 3       | Supply crushed basalt, size 225 to 450mm | m³  |  
  (a) within 14m reach from bank edge  
  (b) for reach between 14m & 25m | 50 quote from Boral Quarries (ref fax dated 31 May 1990)  
  300mm crushed rock $32.68/tonne, equivalent to $49/m³  
  (assuming density when placed of 1.5 t/m³)  
  Use $5/m³ for preliminary costing purposes  
  
  (b) for reach between 14m & 25m |  
  N Scale of Trueway Transport tel advice (049 5159999)  
  o "super-reach" excavator (reach 18.5m) $1280/day  
  o capacity placing rock underwater, based on Williams R experience is 100 t/hr x 10 hrs/day  
  Rate therefore = $1.92/m³  
  (assuming density when placed of 1.5 t/m³)  
  Use $4/m³ for preliminary costing purposes  
  
  (b) for reach between 14m & 25m |  
  N Scale of Trueway Transport tel advice (049 5159999)  
  o "monster-reach" excavator (reach 33m) $2200/day  
  o assume production rate 60% of above  
  Rate therefore = $5.50/m³  
  (assuming density when placed of 1.5 t/m³)  
  Use $7/m³ for preliminary costing purposes |
| 4       | Place rock protection on dressed river bank below low water  
  (a) within 14m reach from bank edge  
  (b) for reach between 14m & 25m | m³  |  
  (a) N Scale of Trueway Transport tel advice (049 5159999)  
  o "super-reach" excavator (reach 18.5m) $1280/day  
  o capacity placing rock underwater, based on Williams R experience is 100 t/hr x 10 hrs/day  
  Rate therefore = $1.92/m³  
  (assuming density when placed of 1.5 t/m³)  
  Use $4/m³ for preliminary costing purposes  
  
  (b) for reach between 14m & 25m |  
  N Scale of Trueway Transport tel advice (049 5159999)  
  o "monster-reach" excavator (reach 33m) $2200/day  
  o assume production rate 60% of above  
  Rate therefore = $5.50/m³  
  (assuming density when placed of 1.5 t/m³)  
  Use $7/m³ for preliminary costing purposes |
| 5       | Supply & place geotextile fabric  
  Dupont Typar 3470 or equivalent | m²  |  
  (a) Width (4.25m) allow 20mms excavator lure plus 1 hr labour @ $25/hr to place underwater at depth 10m  
  Therefore placement cost = $100/(4.25)(10) = $2.35/m²  
  Svy $5/m² for supply & lay  
  
  (b) for reach between 14m & 25m | 5 quote from Boral Quarries (ref fax dated 31 May 1990)  
  Supply cost $918/roll, equiv to $1.44/m²  
  N Scale of Trueway Transport tel advice re placement for std width (4.25m) allow 20mns excavator hire plus 1 hr labour @ $25/hr to place underwater at depth 10m  
  Therefore placement cost = $100/(4.25)(10) = $2.35/m²  
  Svy $5/m² for supply & lay |
| 6       | Supply PVC coated gabion mattress cages (230mm thick), assemble, fill with rock, close and secure, and place underwater | m²  | 125 tel advice from J Wilson (manager, Kypros Cvl Engrs) based on previous work for PWD in Georges R but at shallower depths (tel 604 5222) |
| 7       | Supply PVC coated gabion mattress cages (230mm thick), assemble, place on river foreshore above low water, fill with rock, close and secure | m²  | 60 As above  
  Rate includes $5/m² allowance for ground preparation |
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<td>Supply &amp; install Foreshore Protection grout-filled mattresses, thickness 70 mm</td>
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<td>10</td>
<td>Undertake marine-based landscaping/planting comprising saltmarsh &amp; Aegiceras corniculatum (river mangrove)</td>
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215 Quote from Mitsubishi Aust (ref fax dated 4/6/90) supply cost (FIS Sydney) for nylon coated NSC sheet piling = A$965/t. For 10 mm thickness section, unit weight = 0.12 t/m². Therefore supply cost = $115.80/m²

tel advice from G Workman of Ausmar marine contractors on 28 May 1990 (tel 524 5331) driving cost for depths of <8m approx $80/m², and 8-16m approx $100/m²

50 tel advice from M Roberts of Foreshore Protection P/L (tel 482 1222) based on recent placement of 9000m² to depth of 20m at Garden Island Fleet Base

20 advice from Newcastle office re Throsby Creek project cf also Cordell’s kikuyu turf $6.50/m² + ground prep $0.50/m²

5L native shrubs $10 ea
## 1. ESTIMATE OF QUANTITIES

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**NOTE:** Calculated quantities are approximate only, based on limited survey information. Although quantities in this and subsequent spreadsheets are shown to a large number of significant figures, this is only as a result of spreadsheet arithmetic and should not be regarded as implication of accuracy beyond plus or minus 20%.

## 2. COST ESTIMATE

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EAST HILLS BANK EROSION STUDY
A2 2  OPTION B  SHEETPILING OPTION

1. ESTIMATE OF QUANTITIES

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Say $2,180,000
EAST HILLS BANK EROSION STUDY
A23 OPTION C RENO-MATTRESS OPTION

1. ESTIMATE OF QUANTITIES

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<td></td>
<td>2799</td>
</tr>
<tr>
<td></td>
<td>GRAND TOTAL</td>
<td></td>
<td></td>
<td>8937</td>
<td></td>
<td>12150</td>
</tr>
</tbody>
</table>

2. COST ESTIMATE

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Rate</th>
<th>Quantity</th>
<th>Amount ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undertake excavation within river as per Option A</td>
<td>m3</td>
<td>4</td>
<td>15,118</td>
<td>60,472</td>
</tr>
<tr>
<td>(a) for max reach 14m from bank edge</td>
<td>m3</td>
<td>8</td>
<td>5,775</td>
<td>46,200</td>
</tr>
<tr>
<td>(b) for reach between 14m &amp; 25m</td>
<td>m2</td>
<td>125</td>
<td>12,150</td>
<td>1,518,738</td>
</tr>
<tr>
<td>Supply PVC coated gabion mattress cages (230mm thick), assemble, fill with rock, close &amp; secure, and place underwater</td>
<td>m2</td>
<td>5</td>
<td>8,937</td>
<td>44,685</td>
</tr>
<tr>
<td>Supply &amp; place geotextile fabric Dupont Typar 3476 or equiv</td>
<td>m2</td>
<td>1,670,995</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Say $1,670,000</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
# EAST HILLS BANK EROSION STUDY
## A2.4 OPTION D: GROUT-FILLED NYLON MATTRESS

### 1. ESTIMATE OF QUANTITIES

<table>
<thead>
<tr>
<th>Site No</th>
<th>Cross Section No</th>
<th>Length (m) of bank represented by X-S</th>
<th>Design Elev'n of Base of Toe</th>
<th>Grout-filled mattress length (m)</th>
<th>Quantities area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31A</td>
<td>74</td>
<td>54</td>
<td>14.3</td>
<td>1061</td>
</tr>
<tr>
<td></td>
<td>31</td>
<td>63</td>
<td>63</td>
<td>15.9</td>
<td>1002</td>
</tr>
<tr>
<td></td>
<td>30B</td>
<td>53</td>
<td>69</td>
<td>16.9</td>
<td>889</td>
</tr>
<tr>
<td></td>
<td>30A</td>
<td>50</td>
<td>75</td>
<td>18.1</td>
<td>904</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>77</td>
<td>80</td>
<td>19.0</td>
<td>1454</td>
</tr>
<tr>
<td></td>
<td>29A</td>
<td>87</td>
<td>93</td>
<td>21.2</td>
<td>1838</td>
</tr>
<tr>
<td></td>
<td>29</td>
<td>65</td>
<td>100</td>
<td>22.6</td>
<td>1468</td>
</tr>
<tr>
<td></td>
<td>28A</td>
<td>33</td>
<td>100</td>
<td>22.6</td>
<td>734</td>
</tr>
<tr>
<td>subtotal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>9351</td>
</tr>
<tr>
<td>2</td>
<td>24</td>
<td>36</td>
<td>40</td>
<td>11.8</td>
<td>424</td>
</tr>
<tr>
<td></td>
<td>23A</td>
<td>78</td>
<td>49</td>
<td>13.5</td>
<td>1043</td>
</tr>
<tr>
<td></td>
<td>23</td>
<td>64</td>
<td>60</td>
<td>15.4</td>
<td>985</td>
</tr>
<tr>
<td></td>
<td>22A</td>
<td>23</td>
<td>60</td>
<td>15.4</td>
<td>346</td>
</tr>
<tr>
<td>subtotal</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2799</td>
</tr>
<tr>
<td>GRAND TOTAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>12150</td>
</tr>
</tbody>
</table>

**NOTE**: Calculated length of mattress per cross-section allows for toe beyond collapsible hinge, equivalent to 30m vertical depth

### 2. COST ESTIMATE

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Rate</th>
<th>Quantity</th>
<th>Amount ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undertake excavation within river as per Option A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) for max reach 14m from bank edge</td>
<td>m³</td>
<td>4</td>
<td>15118</td>
<td>60,472</td>
</tr>
<tr>
<td>(b) for reach between 14m &amp; 25m</td>
<td>m³</td>
<td>8</td>
<td>5775</td>
<td>46,200</td>
</tr>
<tr>
<td>Supply &amp; install Foreshore Protection grout-filled mattresses</td>
<td>m²</td>
<td>50</td>
<td>12150</td>
<td>607,495</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>714,167</td>
</tr>
</tbody>
</table>

Say $720,000
**EAST HILLS BANK EROSION STUDY**

**A2.5 "SOFT" & "HARD" FINISH**

**ESTIMATE OF QUANTITIES**

1 **SOFT FINISH**

<table>
<thead>
<tr>
<th>Site No</th>
<th>Cross Section No</th>
<th>Length (m) of bank represented by X-S</th>
<th>Topsoil stripping, stockpiling &amp; regrading area (m²)</th>
<th>Excavate on river foreshore area (m²)</th>
<th>vol (m³)</th>
<th>vol (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>31A</td>
<td>74</td>
<td>18</td>
<td>133</td>
<td>36</td>
<td>346</td>
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<tr>
<td></td>
<td>31</td>
<td>63</td>
<td>24</td>
<td>151</td>
<td>47</td>
<td>385</td>
</tr>
<tr>
<td></td>
<td>30B</td>
<td>53</td>
<td>12</td>
<td>63</td>
<td>20</td>
<td>126</td>
</tr>
<tr>
<td></td>
<td>30A</td>
<td>50</td>
<td>24</td>
<td>120</td>
<td>81</td>
<td>609</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>77</td>
<td>24</td>
<td>184</td>
<td>110</td>
<td>1331</td>
</tr>
<tr>
<td></td>
<td>29A</td>
<td>87</td>
<td>30</td>
<td>260</td>
<td>90</td>
<td>1142</td>
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<tr>
<td></td>
<td>29</td>
<td>65</td>
<td>30</td>
<td>195</td>
<td>61</td>
<td>516</td>
</tr>
<tr>
<td></td>
<td>28A</td>
<td>33</td>
<td>18</td>
<td>58</td>
<td>39</td>
<td>170</td>
</tr>
<tr>
<td>subtotal</td>
<td></td>
<td>500</td>
<td>1164</td>
<td>4024</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2 **HARD FINISH**

<table>
<thead>
<tr>
<th>Site No</th>
<th>Cross Section No</th>
<th>Length (m) of bank represented by X-S</th>
<th>Topsoil stripping, stockpiling &amp; regrading area (m²)</th>
<th>Excavate on river foreshore area (m²)</th>
<th>vol (m³)</th>
<th>vol (m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>24</td>
<td>36</td>
<td>12</td>
<td>43</td>
<td>60</td>
<td>346</td>
</tr>
<tr>
<td></td>
<td>23A</td>
<td>78</td>
<td>24</td>
<td>186</td>
<td>77</td>
<td>881</td>
</tr>
<tr>
<td></td>
<td>23</td>
<td>64</td>
<td>18</td>
<td>115</td>
<td>64</td>
<td>622</td>
</tr>
<tr>
<td></td>
<td>22A</td>
<td>23</td>
<td>24</td>
<td>54</td>
<td>60</td>
<td>189</td>
</tr>
<tr>
<td>subtotal</td>
<td></td>
<td>200</td>
<td>398</td>
<td>2038</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

GRAND TOTAL 700 1562 6662

2 **"HARD" FINISH**

Total quantity of reno-mattress = (700)(40) m²

= 2800 m²
## CALCULATION OF QUANTITIES

### 1 SOFT FINISH

<table>
<thead>
<tr>
<th>Item Description</th>
<th>Unit</th>
<th>Rate ($/m³)</th>
<th>Quantity</th>
<th>Amount ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strip &amp; stockpile topsoil (min 300mm) inland of existing revetments, then spread &amp; regrade</td>
<td>m³</td>
<td>21</td>
<td>1562</td>
<td>32,810</td>
</tr>
<tr>
<td>Undertake excavation on river foreshores</td>
<td>m³</td>
<td>4</td>
<td>6662</td>
<td>26,648</td>
</tr>
<tr>
<td>Undertake marine-based landscaping/planting comprising saltmarsh &amp; Aegiceras corniculatum (river mangrove) on inter tidal beach</td>
<td>m²</td>
<td>20</td>
<td>4970</td>
<td>99,400</td>
</tr>
</tbody>
</table>

Undertake excavation on river foreshores and assemble, place on river foreshore above low water, fill with rock, close & secure

### 2 "HARD" FINISH

<table>
<thead>
<tr>
<th>Item Description</th>
<th>Unit</th>
<th>Rate ($/m²)</th>
<th>Quantity</th>
<th>Amount ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Supply &amp; place geotextile fabric &lt;br&gt;Dupont Tyrap 3470 or equivalent</td>
<td>m²</td>
<td>5</td>
<td>2800</td>
<td>14,000</td>
</tr>
<tr>
<td>Supply PVC coated gabion mattress cages &lt;br&gt;(230mm thick), assemble, place on river foreshore above low water, fill with rock, close &amp; secure</td>
<td>m²</td>
<td>60</td>
<td>2800</td>
<td>168,000</td>
</tr>
</tbody>
</table>

Supply PVC coated gabion mattress cages <br>(230mm thick), assemble, place on river foreshore above low water, fill with rock, close & secure

---

Say $160,000

Say $180,000
FIGURES
Figure 1.1
LOCALITY PLAN
EAST HILLS GOLF COURSE
Sydney, Australia.

Proposed Concept for Redevelopment to INTERNATIONAL STANDARD GOLF COMPLEX

This plan is exclusive only and under the approval of the architect and engineer.

Prepared by:
CHYUDA AUSTRALIA Pty Ltd

Figure 1.3

Development Summary
- Every Road Inner Land
- Proposed Course Length
- Practice Fairways
- 9-Hole Par Course
- Water Bodies
- Foreshore Reserve (subject to survey of land)

Development Area

PROPOSED REDEVELOPMENT PLAN
FOR EAST HILLS GOLF COURSE

Figure 1.3
1926 SURVEY PLAN
Figure 2.1 Sheet A
Figure 2.2

1959 SURVEY PLAN

APPROXIMATE SCALE 1:9600
NOTES
1 Cross section numbers correspond to cross sections from PWD Hydrosurvey of 1976.
Chamage of CS 21 was km 25.25 in survey of 1976.

2 For cross sections refer to Figure 2.6
Sheets A to G

LEGEND
km+6  Chamage along Georges River (km)
-----  Location of Thalweg (1984)

RIVER CHAINAGE AND
LOCATION OF CROSS SECTIONS
Figure 2.3
Figure 2.4

NOTES
1. Only floods greater than 5m at Liverpool are shown.
2. Flood heights prior to 1940 are very doubtful.
3. For floods that occurred prior to 1950 the month is not known.
4. This record may be incomplete.

FLOOD HEIGHT (m A H D)

1964
- JUNE
- AUGUST
- APRIL
- NOVEMBER

1970
- PHOTOGRAMMETRY (JULY 1970)

1980
- JUNE
- JULY
- MARCH

1990
- AUGUST
- APRIL
- PHOTOGRAMMETRY (NOV 1988)

1910
- HYDRO SURVEY (MAY 1978)

1937
- HYDRO SURVEY (SEPT 1976)

1940
- HYDRO SURVEY (APRIL 1977)

1950
- Regimented (MAY 1978)

1960
- HYDRO SURVEY (OCT 1984)

1970
- HYDRO SURVEY (NOV 1959)

1980
- HYDRO SURVEY (NOV 1959)

1990
- HYDRO SURVEY (NOV 1959)

1960
- HYDRO SURVEY (FEB 1926)

1970
- PHOTOGRAMMETRY (FEB 1926)

1980
- PHOTOGRAMMETRY (FEB 1926)

1990
- PHOTOGRAMMETRY (FEB 1926)
NOTE: "Thalweg" is line of greatest depth along the channel.
NOTE
Cross Section Profiles of surveys marked with a cross (x) have been adjusted laterally to achieve an appropriate fit.

NOTES
1. Cross Sections are drawn from left to right looking downstream.
2. Cross sectional areas given are for the 1984 survey below Om Standard Datum.
**1959 SURVEY**

**CROSS SECTION No. 29**

- **Former Cross Section No. 29**
- **80 metres upstream**
- **Area = 360m²**

**NOTES**

- Cross Section Profiles of surveys marked with a cross (×) have been adjusted laterally to achieve an approximate fit.

---

**1959 SURVEY**

**CROSS SECTION No. 29**

- **Former Cross Section No. 29**
- **120 metres downstream**
- **Area = 660m²**

---

**Area = 350m²**

**C.S. No. 27**

**810 m**

**NOTES**

1. Cross Sections are drawn from left to right looking downstream.
2. Cross sectional areas given are for the 1984 survey below 0m Standard Datum.

---

**LEGEND**

- **SURVEY OF 1959**
- **SURVEY 1 (1976)**
- **SURVEY 2 (1977)**
- **SURVEY 3 (1978)**
- **SURVEY 5 (1984)**

---

**CROSS SECTIONS**

**CS 27 to CS 29**

**Figure 2.6 Sheet C**
Cross Sections
CS 30 to CS 32
Figure 2.6  Sheet D
CROSS SECTIONS
CS 33 to CS 35
Figure 2.6 Sheet E
NOTE
Cross Section Profiles of surveys marked with a cross (x) have been adjusted laterally to achieve an approximate fit.

CAUTION
This plan has been prepared at a scale of 100m = 1mm. It is not to be scaled for other purposes. The accuracy of this plan is subject to all other limitations not stated. See Plan Cat W 8756 for Index Plan.

Figure 2.6 Sheet F
Area = 305m²

100m BELOW STANDARD DATUM

LEGEND


NOTE

1 Cross Sections are drawn from left to right looking downstream
2 Cross sectional areas given are for the 1984 survey below 0m Standard Datum

CROSS SECTIONS CS39 to CS41
Figure 2.6 Sheet G
Fig. 2.7

TIDAL GRADIENTS AND LAGS
1979/1980

NOTES
1 Datum is Indian Spring Low Water (Ocean) which is 0.89 m below Standard Datum
2 Gradients are from an Harmonic Analysis of PWD and Fort Denison Records 12th December 1979 to 9th January 1980
PORTION OF 1961 AERIAL PHOTOGRAPHY
Figure 3.3
NOTE: For Sheets A, B, C & D see Figure 3.6
Sheets A to D respectively.
LEGEND

- 1988
- 1978
- 1970
- 1949
- 1930

360 m Corresponds to location of profile used in linear regression analysis.

--- Natural or man-made levee.

---- Approximate scarp location due to thick vegetative cover.

SCARP LOCATION COMPARISONS

km00 to km0.8
Figure 3.6 Sheet A
LEGEND

- 1988
- 1978
- 1970
- 1949
- 1930

Corresponds to location of profile used in linear regression analysis.

Natural or man-made levee.

Approximate scarp location due to thick vegetative cover.

SCARP LOCATION COMPARISONS
km 0.8 to km 1.4
Figure 3.6 Sheet B
Corresponds to location of profile used in linear regression analysis.

Natural or man-made levee.

Approximate scarp location due to thick vegetative cover.

SCARP LOCATION
COMPARISONS
km 1.4 to km 2.0
Figure 3.6 Sheet C
SCARP LOCATION COMPARISONS
km 2.0 to km 2.8
Figure 3.6 Sheet D

LEGEND

- Natural or man-made levee.
- Approximate scarp location due to thick vegetative cover.

1988
1978
1970
1949
1930
EAST HILLS BANK EROSION STUDY
GEORGES RIVER
LEFT BANK

NOTES
1. All data obtained from photogrammetric analysis.
2. 1930 is the baseline (i.e. 0m line) and subsequent years are cumulative, e.g. 1978 line shows 48 years movement after 1930
EAST HILLS BANK EROSION STUDY
GEORGES RIVER

LEFT BANK

Period:
1930-88
1930-49
1949-70
1970-78
1978-88

NOTE: All data obtained from photogrammetric data

RIGHT BANK
NOTES

1. This analysis was undertaken from scarp location data obtained photogrammetrically using a Wild AC1 stereo-restitution instrument.

2. Results are presented for scarp movements at selected profiles at Site 1 and Site 2. The rate of scarp movement was calculated using linear regression analysis for all dates of photography - see Figures 3.10 and 3.11.

REGRESSION ANALYSIS
SITE 1 and SITE 2
Figure 3.9
NOTES
1. These graphs show, for each profile location, the change in scarp position for all dates of photography. Distances are measured relative to the 1930 scarp location. Positive values represent erosion relative to the 1930 scarp position.
2. The line plotted is the result of linear regression analysis of all dates of photography.
3. The slope of the line represents the historical trend in scarp movement. E.g. A slope of 0.2 represents an average erosion rate of 0.2 m/year.
4. Locations of profiles are shown on Figure 3.6 Sheets B and C.

LINEAR REGRESSION PLOTS
SITE 1
Figure 3.10
**CHAINAGE 140**
**LEFT BANK**

**SLOPE = 0.20, R SQR = 0.89**

**CHAINAGE 280**
**LEFT BANK**

**SLOPE = 0.05, R SQR = 0.66**

**CHAINAGE 200**
**LEFT BANK**

**SLOPE = 0.10, R SQR = 0.55**

**CHAINAGE 320**
**LEFT BANK**

**SLOPE = 0.13, R SQR = 0.95**

**CHAINAGE 240**
**LEFT BANK**

**SLOPE = 0.12, R SQR = 0.63**

**CHAINAGE 360**
**LEFT BANK**

**SLOPE = 0.12, R SQR = 0.81**

**NOTES**

1. Refer to notes 1, 2 & 3 as shown on Figure 3.10
2. Locations of profiles are shown on Figure 3.6 Sheet A.

**LINEAR REGRESSION PLOTS**

**SITE 2**

**Figure 3.11**
LOCATION OF CROSS SECTIONS
USED FOR
QUANTITY CALCULATIONS
Figure 4.1
**OPTION A**
PLACED ROCK

8 m$^3$/m of additional rock at toe as underscour protection

**OPTION B**
SHEET PILING

Sheet pile with wall thickness 10.5 mm
Depth varies from 12 m to 16 m

**OPTION C**
RENO-MATTRESS

**OPTION D**
GROUT-FILLED NYLON MATTRESS

Collapsible hinge to prevent progression of underscouring

**NOTE:** Protection above low water not shown. Refer to Figure 4.3 for details

**FORESHORE PROTECTION OPTIONS**
BELOW LOW WATER

Figure 4.2
Minimal encroachment beyond top of existing bank

Existing surface level

Reno - mattress

1 1/2

MEAN HIGH WATER

0.23

MEAN LOW WATER

“HARD” FINISH

10m Maximum

Mangroves

Saltmarsh

Existing surface level

Anchor mat (open mesh)

MEAN HIGH WATER

MEAN LOW WATER

“SOFT” FINISH

NOTE Protection below low water not shown Refer to Figure 4.2 for details

FORESHORE PROTECTION OPTIONS

ABOVE LOW WATER

Figure 4.3
Plate 1
23 January 1990
Riverbank at Site 1 showing low-tide bench and phragmite reeds (left bank, at km 1.5 looking upstream)

Plate 2
23 January 1990
Eroding bank at Site 1 showing material deposited on low-tide bench. Note reduction in low-tide bench width further into the bend (left bank, at km 1.5 looking downstream)

Plates 1 & 2
Plate 3
Riverbank at Site 2 showing makeshift jetty and exposed tree roots (left bank, at km 0.3 looking downstream)

Plate 4
Eroding bank at Site 2 showing past piecemeal protection (concrete slabs) on low-tide bench (left bank, at km 0.3 looking upstream)