CLEARY BROS (BOMBO) PTY LTD

GERROA SAND MINE: FLOOD STUDY UPDATE

BLUE ANGLE CREEK FLOOD STUDY

Sand Mine Extension

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Prepared by:

Evans and Peck Pty Ltd
20/390 Eastern Valley Way
East Roseville NSW 2069

Tel: (02) 9882 0500
Fax: (02) 9417 8850
Email: sperrens@evanspeck.com

Paterson Consultants
PO Box 596
Grafton NSW 2460

Tel: (02) 6643 1588
Fax: (02) 6642 7566
E-mail: patcon@nor.com.au

SMEC
PO Box 1052
North Sydney NSW 2059

Tel: (02) 9925 5555
Fax: (02) 9925 5566
E-mail: smecsyd@smec.com.au

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1. INTRODUCTION

1.1 BACKGROUND

This report has been prepared to assess the impact on flooding levels as part of the development application for the current (2005) proposed sand mine extension.

Hydrologic and hydraulic modelling has been carried out for a previous extension to the sand mine. The results of this modelling were presented in the Flood Study report prepared by Perrens Consultants in 2003. This current modelling is based on the previous modelling updated to include the current proposed extension.

A plan showing the location of the proposed development is contained in **Figure 1.1**.

1.2 STUDY SCOPE

This Flood Study has been prepared to assess the impacts on flooding of Cleary Bros 2005 proposed sand mine extension. The Flood Study includes the Crooked River and Blue Angle Creek catchments and provides estimates of flood levels at the site for a range of floods from the 2 year ARI up to the Probable Maximum Flood (PMF). The following scenarios have been assessed:

- 2002 mine topography;
- 2005 existing mine topography;
- proposed extension.

The flood study brings together three separate areas of investigation and analysis:

- estimates of flood flows from the catchments of Foys Swamp and Crooked River;
- analysis of ocean level conditions and the beach “breakout” process;
- the hydraulic analysis to determine flood levels that result from various combinations of flood flows, ocean levels and beach “breakout” conditions.

This report provides an assessment of the flood regime and flood levels affecting Foys Swamp in general and the proposed development site in particular.

The flood levels derived as a result of this flood study form the basis for assessing the impact of the sand mine operation. Accordingly, we have carried out the analysis in sufficient depth, and with appropriate rigour, to ensure that the flood level estimates and the assessment of the impacts of the development are robust.

1.3 CATCHMENT DESCRIPTION

The catchment of the Crooked River, including Blue Angle Creek, has a total area of approximately 32 km^2. It comprises a mosaic of native forest areas and pasture areas which have been cleared of all original vegetation. The flood producing characteristics of the catchment are one of the factors which determine flood levels occurring along the Crooked River.

The Crooked River catchment is characterised by very steep slopes around the edge of the valley, with fairly flat v-shaped valleys leading to flat and wide floodplains. Crooked River runs from a number of tributaries on the southern side of Currys Mountain in a south-easterly direction towards Gerroa.

The main channel of the Crooked River is relatively shallow and the mouth is almost completely closed by a sand bar. The sand bar is unstable and is breached under conditions of storm surge or river flooding. The entrance conditions at the mouth of the river are a key factor determining flood levels in the river. In the absence of flood conditions the water level in the river will primarily be controlled by sea level but will be elevated slightly above sea level because of the hydraulic retardance of the entrance bar.

An extensive area of low lying swamps and swales borders the Crooked River along its lower reaches. These areas which are generally at an elevation of less then 2 m AHD, have slopes of less than 0.5% and are characterised by poor draining soils with a high water table at or near the surface. These areas provide a flood fringe storage area in which some flood volume is stored but which provide minimal flood conveyance because of shallow flow and high hydraulic resistance.

The Blue Angle Creek catchment forms a sub-catchment within the larger Crooked River catchment. The edge of the catchment is characterised by some steep slopes particularly to the west, however the majority of the land is comparatively flat.

Blue Angle Creek runs almost parallel to the Bolong Road behind Seven Mile Beach. Upstream of the creek is a large swampy area known as Foys Swamp. The total area of the Blue Angle Creek catchment is around 12 km^2.

1.4 FLOODING CONDITIONS IN FOYS SWAMP

Flooding in the Foys Swamp area can occur as a result of a number of processes and the interaction between those processes:

- Runoff from the catchment surrounding Foys Swamp that tends to pond within the swamp because of the restricted capacity of Blue Angle Creek.
- Flooding in the Crooked River estuary as a result of rainfall on the Crooked River catchment. Because the land level within Foys Swamp is very low, floodwater from Crooked River can back up into Foys Swamp. Such backup is, however, hindered by a culvert with flap gates on an access road from Seven Mile Beach Road (refer **Figure 1.1**).
- Flooding in Crooked River Estuary as a result of elevated ocean levels caused by the state of the tide and the presence of a depression of the coast which leads to a storm surge and “wave setup” as a result of strong onshore winds.

While it is likely that the rainfall that leads to flooding in Foys Swamp will also lead to flooding in Crooked River, the peak flood levels from these sources will not necessarily occur at the same time. In addition, because Foys Swamp has a very large storage capacity relative to the surrounding catchment, it would be expected that the range of flood levels between common and rare floods would be relatively minor.

The peak flood level reached in Crooked River will be strongly influenced by the state of the beach berm at the mouth when a flood occurs. In a major flood, the floodwater will scour a channel through the sand. The rate of development of the scour channel and the final size of the channel will govern the peak flood level in Crooked River. The process of scour is, in turn, governed by the ocean level at the time that the flood occurs. Again, the relative timing of the peak ocean level and the peak flood flow will dictate the resulting flood level in Crooked River and Foys Swamp.

1.5 METHODOLOGY

The following general methodology, together with the respective section/appendix in this report, was adopted for this study:

1.	Undertake data collection and review, including existing studies.	Section 2 Appendix A
2.	Commission a survey to obtain additional cross-sectional information to obtain further topographic definition in the area of the proposed extension for input to the hydraulic model.	Section 2
3.	Set-up a hydrologic model using RAFTS-XP to generate design flows for input to the hydraulic model.	Section 3 Appendix B
4.	Analyse tailwater conditions and identify a set of suitable tail water levels for input to the MIKE-11 model.	Section 4 Appendix C
5.	Update hydraulic model, using MIKE-11, to determine design water levels.	Section 5 Appendix D
6.	Compare model results with historic flood information to assess results.	Section 5 Appendix D
7.	Produce estimates of flood levels at key locations.	Section 5 Appendix D

2. DATA COLLECTION AND REVIEW

2.1 PREVIOUS STUDIES

2.1.1 Flood Studies

A number of flood studies have been undertaken in the past for various purposes:

- Perrens Consultants prepared the Blue Angle Creek Flood Study for the 2003 Sand Mine Extension. Note that the Perrens Consultants staff who prepared the 2003 Flood Study are now at Evans & Peck and have produced this current Flood Study.
- As part of an urban land capability study, GHD prepared a preliminary flood study in 1989. That study provided indicative 100 year Average Recurrence Interval (ARI) flood levels in land on the northern and eastern side of the Crooked River Estuary which was being considered for urban development.
- GHD prepared two further flood study reports in connection with the Gerringong-Gerroa sewerage scheme in 1999. These reports examined flood levels in the Crooked River estuary and along Blue Angle Creek for purposes of defining the level at which critical elements of the sewage treatment plant should be set.

The latter flood studies prepared by GHD took a relatively simplistic and highly conservative approach to the estimation of flood levels. Shortcomings in the approach included:

- the analysis ignored any difference in timing of flood peaks in Crooked River and Foys Swamp and assumed that the peak flows occurred at the same time;
- the analysis also ignored any effects of scouring at the mouth of the estuary and assumed that the peak flood flows occurred at exactly the same time as the peak ocean level;
- a further element of conservatism was included by assuming that the 100 year ARI peak flow occurred at exactly the same time as the peak ocean level associated with a high tide;
- the analysis does not adequately account for flood storage effects within Foys Swamp and on the Crooked River floodplain. The effect of storage of flood water will be to lower the predicted flood levels.

The effect of compounding all these assumptions is to provide flood level estimates that are conservative. Consequently, the 100 year ARI flood level estimated by GHD is actually likely to be rarer than once in 100 years on average.

2.1.2 Other Reports

A number of other reports were identified as being of possible interest to the study, including:

- Local Environment Study – Blue Angle Creek and Crooked River
  Quality Environmental Management, 1992
- Coastal Hazards Analysis and Sand Resource Assessment for Gerroa. Draft LEP
  Webb McKeown and Associates, 1992
- Extreme Ocean Water Levels – Shoalhaven River
  Lawson & Treloar, 1987
- Crooked River Data Compilation Study
  Kiama Council, date unknown

The first three reports were obtained and the limited information of relevance was extracted. Unfortunately, Kiama Municipal Council was unable to locate a copy of the last item for use in this study.

2.2 MAPPING

1:25,000 topographic mapping with 10 m contour intervals were available for the study area. In addition, an aerial ortho-photo map at 1:2,500 scale with 1.0 m contour intervals for the development site was provided by Cleary Bros.

2.3 RAINFALL DATA

2.3.1 Historic Data

Ranked 1 day, 2 day and 3 day rainfall data was required for the tail water level analysis (refer **Appendix C**). Daily rainfall data was obtained from the Bureau of Meteorology. Available stations within and adjacent to the catchment are listed in **Table 2.1** below and their location shown on **Figure 1.1**.

Table 2.1
BoM Rainfall Stations

Station Number	Station Name	Period of Record
68027	Mayflower Village (Gerringong)	1895 -
68038	Kiama Bowling Club	1897-
68175	“Nyora”, Toolijooa	1967-
68197	Foxground Road	1945 – 2001

The rainfall analysis was primarily based on data from the station at Gerringong. Missing data was infilled based mainly on correlation with data from Kiama Bowling Club. Where necessary, disaggregation of accumulated data was carried out based on the patterns within the other stations.

The Manly Hydraulics Laboratory was also contacted regarding rainfall records, however no data is collected by that organisation within the study area.

2.3.2 Design Data

Design storms for a range of frequencies up to the 100 year ARI were derived using principles given in Book II of Australian Rainfall and Runoff (1998).

Temporal patterns for various zones in Australia are presented in Volume 2 of ARR and are in-built in the RAFTS model.

The Probable Maximum Precipitation (PMP) was estimated using the Generalised Short-Duration Method (GSDM) (Bulletin 53, BoM, 1994) as recommended in Book IV of Australian Rainfall and Runoff (1997).

2.4 HISTORIC FLOOD LEVELS

Attempts were made to obtain historic flood level information to assist with the calibration of the hydraulic model. A number of possible sources were identified and contacted, including local authorities and land holders. Rail Infrastructure Corporation and Fleur Daniel were identified as possible sources of information regarding flooding of the Illawarra railway line between Berry and Gerringong. Very little useful data was obtained through this process, as outlined below.

Kiama Municipal Council
- no flood records kept;
- no anecdotal information on flooding.

DLWC
- no flood records obtained as Kiama Council not part of DLWC's Flood Program.

Local Caravan Parks
- Gerroa Shore Caravan Park (located on northern side of Crooked River) – no flooding in last 4 years;
- Seven Mile Beach Holiday Park (located on southern side of Crooked River) – no flooding records.

Rail Infrastructure Corporation (RIC)
- flood record cards cannot be located, possibly with Fleur Daniel;
- original working plans show only proposed construction, not as-built;
- bridges and culverts have been upgraded over last 20 years, so any flood levels collected probably not relevant.

Fleur Daniel
- no information.

Cleary Bros.
- limited information regarding flood levels kept;
- anecdotal evidence of a flood level in Foys Swamp of 2.5 m in February 2002;
- anecdotal evidence that flooding of the Cleary Bros. access road over Blue Angle Creek has never occurred over the 20 year period since the construction of the floodgates (shown on **Figure 1.1**);
- photographs of flooding in February 2002.

2.5 BEACH BERM CONDITIONS

An analysis of the beach berm conditions at the mouth of the Crooked River was undertaken (refer **Appendix C** for more details). This included:

- a detailed inspection of beach berm conditions and hydraulic conditions in Blue Angle Creek;
- discussions with DIPNR to establish proposed future operational rules for "artificial" opening of the entrance of Crooked River;
- sampling of sand from the berm at the mouth of Crooked River in order to determine its scouring characteristics.

Discussions were held with Kiama Municipal Council to establish historic operational rules for "artificial" opening of the entrance of Crooked River. It was found that the entrance was not opened very frequently and was only considered when water entered the caravan park on the northern side of the river. Any opening of the river mouth was constrained by Acts administered by DIPNR and the NSW Fisheries, and was therefore subject to their permission. Kiama Municipal Council has not kept any records of when the entrance has been artificially opened.

Additional anecdotal evidence regarding the opening of the Crooked River entrance were obtained from Mr Colin Stoddart, Town Planner and resident of Gerringong, based on ten years of records and observations. This information is contained in **Appendix A** and a summary provided below.

1. *River **opened** by Kiama Municipal Council late **November 1990***
   - *river had been closed for some time*
   - *fish had gathered at the entrance trying to escape*

2. *River **opened late May 1995***
   - *possibly closed over extended period over warmer months*
   - *poor water quality with water becoming turbid and algae present.*

3. *Possible **closure mid-April 1996 – late May 1996***

4. *River **opened** about **12/7/1997** after closure of 6 – 8 weeks.*

5. *During **1998 and 1999** the river closed very briefly a number of times for a few days only. During autumn/winter 1998 the channel was very narrow and was almost closed (it closed briefly at least once). In late 1999 after exhibition of the EIS for the sewage treatment plant the river was observed to close briefly at least twice.*

6. *Entrance **closed** again approximately **mid-June 2002.***
   Of the closures, three were opened after assistance from Kiama Council (mechanical opening). In addition, Kiama Council carried out major sand removal just to the west of the footbridge over an extended period in about 1996.

2.6 SURVEY

Topographic survey to define the flood storage and conveyance characteristics of Foys Swamp, Crooked River and Blue Angle Creek was originally obtained in 2002. The survey was used as the basis for the cross-sections of the hydraulic model. Survey of the existing bunding for the sand mining operations was also obtained as well as the dimensions of all relevant hydraulic structures.

As part of the current Flood Study, additional survey was obtained in the vicinity of the proposed 2005 extension.

Figure 1.1 shows the survey obtained for the study, including the additional sections obtained for this assessment of the current proposed sand mine extension.

3. HYDROLOGIC ANALYSIS

The hydrologic computer modelling program RAFTS-XP (XP Software, 1996) was used to simulate the runoff characteristics of the Blue Angle Creek and Crooked River catchments. This model was utilised by GHD for previous studies and was adopted for this present study in the interests of consistency. Full details of the hydrologic modelling are contained in **Appendix B**.

3.1 METHODOLOGY

The RAFTS hydrologic model is a method of converting rainfall on the catchment into runoff. The model subdivides the catchment into a number of sub-catchments. For each sub-catchment, the model adds the runoff into the creek from the sub-catchment to the flow from upstream. In calculating the flow contributed from upstream, the model takes account of the effect of channel and floodplain storage on the shape of the flood hydrograph. The output of the model comprises a full flood hydrograph at the outlet of each sub-catchment which allows the results from the model to be used to assess flow conditions throughout the catchment.

The hydrologic model was configured to be compatible with the hydraulic model, which required full flow hydrographs from catchments draining into Foys Swamp and Crooked River (rather than just the peak flow as used by GHD). The full range of events from the 2 year ARI to the PMF were modelled.

The "tuning" of the RAFTS model involved adjusting the model parameters in order to match peak flows calculated using the Probabilistic Rational Method (PRM) (refer **Appendix B**) as there were no recorded hydrographs against which to calibrate the model. Tuning was achieved by altering the model parameters which govern infiltration losses on the land surface.

The catchment subdivision was chosen to coincide with points of interest in the hydraulic analysis. **Figure 3.1** shows the layout of the hydrologic model, together with the locations of the reference points for input to the hydraulic model.

3.2 ADOPTED MODEL PARAMETERS

Detailed discussion of the model parameters adopted for the model is contained in **Appendix B**. Parameters input to the model include sub-catchment areas and roughness factors, design rainfalls, temporal patterns and rainfall losses.

An initial loss of 10 mm and a continuing loss of 2.5 mm/h was adopted for all storms. The initial rainfall loss is the capacity which must be satisfied before runoff can occur. The continuing rainfall loss then occurs at a constant rate over each time period.

The design storm analysis was based on a "Bx" value of 1.0. Bx is the Storage Coefficient Multiplication Factor, which is used to modify the calculated Storage Time Delay Coefficient (B). The default value for Bx is 1.0. Bx uniformly modifies all sub-catchment B values, thereby attenuating the peak discharge but not altering the total volume of the hydrograph generated.

3.3 DESIGN FLOWS

The RAFTS hydrologic model was run for the 2, 5, 10, 20, 50 and 100 year ARI storms as well as the PMF and for durations ranging from 2 hours to 48 hours. The critical storm, in terms of peak flow at the catchment outlet was found to be the 2 hour event. The flows at Point F (refer **Figure 3.1**) for the various durations are presented in **Table 3.1** below.

Table 3.1
RAFTS Flows (m^3/s), Crooked River Outlet

Duration (h)	20 year ARI	100 year ARI
2	**360**	**535**
4.5	330	470
6	360	490
9	345	390
12	300	405
18	245	335
24	290	380
36	260	345
48	265	355

Table 3.2 presents the peak flows derived from the present RAFTS analysis, together with those presented in the GHD report and the PRM analysis outlined in Appendix B. The flows have been generated for the Crooked River upstream of the confluence with Blue Angle Creek. It was considered that this was the most appropriate location to carry out the comparison, as the PRM analysis is not capable of taking into account the storage effects of Foys Swamp.

Table 3.2 also contains the results for the RAFTS model at the same location but with Bx set at 2.3, to facilitate comparison with GHD's results. The RAFTS model results are for a 2 hour duration storm (20 year and 100 year ARI) and 6 hour storm for the PMF.

Table 3.2
Comparison of Design Flows, Crooked River upstream of Blue Angle Creek

Design Storm ARI (years)	Peak Discharge (m^3/s)			
	RAFTS model		GHD	PRM
	Bx = 1	Bx = 2.3	Bx = 2.3?	
20	255	110	135	310
100	380	165	190	485
PMF	840	695	640	-

It can be seen that the RAFTS model used for this study produces higher flow estimates than those adopted by GHD. The PRM analysis resulted in even higher flows than the RAFTS model.

It appears that the GHD model resulted in lesser flows due to the value adopted for Bx, the storage coefficient. Although not actually documented in their report, it appears that GHD have adopted a value for Bx of 2.3, while this current study has used a value of 1.0. It can be seen that when the current model is run with Bx set to 2.3, the results approach those achieved by GHD. In addition, GHD used a slightly higher initial loss of 12 mm, compared with the 10 mm adopted for the present study.

It is considered that the adoption of the higher flows, generated using a Bx value of 1.0, is appropriate for this present study because:

- there is no justification to use a Bx value other than 1.0;
- the peak flows generated approach those produced using the PRM;
- alteration of Bx does not alter the volume of the hydrograph generated and this study is interested in identifying the volumes producing peak flood levels in Foys Swamp.

Subsequent hydraulic modelling (refer **Section 5**) showed that the 2 hour event was not the critical storm in terms of peak flood levels in Foys Swamp and at the development site. This was found to be the 36 hour event. Hydrographs were prepared for the 5, 10, 20, 50, 100 year ARI 36 hour duration storm and the 6 hour PMF at all of the reference locations shown on **Figure 3.1**, for input to the MIKE-11 model.

4. OCEAN TAIL WATER CONTROL

The Crooked River enters the ocean at the northern end of Seven Mile Beach, adjacent to Black Head and the small township of Gerroa (refer **Figure 1.1**). The entrance channel is trained on its northern side against Black Head and, generally, the inlet is open to the sea. However, the channel is closed intermittently by the transport and deposition of littoral drift from Seven Mile Beach. The estuary, therefore, may be categorised as one of the Intermittently Closed or Open Lakes or Lagoons (ICOLL).

For any given rainfall event, the extent of flooding in the Crooked River catchment and its tributaries will be influenced by, inter alia, the state of the ocean inlet (whether open or closed) and the prevailing meteorological and tidal conditions. Whether the estuary entrance stays open or is closed will depend on the relative magnitudes of the littoral drift transport (and, hence, nearshore wave conditions) to the inlet, acting to close the entrance, and the ebb tidal discharge, which may be enhanced by precipitation, acting to keep the channel open.

Should the inlet be open to the sea then the ocean levels will provide a tailwater control on the discharge of estuary floodwaters. In the nearshore zone these levels will be influenced not only by the tidal stage and storm surge but also by nearshore wave and wind setup. Should the inlet be closed, the crest of the beach berm will present as a weir controlling the downstream water level for floodwater discharge. However berm levels are low and wave action on elevated water levels will cause overtopping.

Previous studies for the Crooked River, which were carried out for the Gerringong-Gerroa Sewerage Scheme project, were undertaken by GHD (1999). That work adopted what was identified as a conservative assumption of combining a 100 year ARI precipitation with a 100 year ARI sea level, the latter being one adopted from a study by Lawson and Treloar (1987) for the Shoalhaven River entrance.

As part of this present Flood Study, SMEC have carried out a more detailed study that explores not only the coincidence of extreme precipitation and ocean wave and water level conditions but also water levels for various inlet configurations. This report is contained in full in **Appendix C** and includes:

- Correlation of dates of significant rainfall to ocean storm events (as defined by “Ocean Storms on the NSW Coast”, Blair, Bremner and Williams and updates).
- Prediction of wave heights off shore based on ocean storm data.
- Transfer of ocean wave data to wave information at the break out of Crooked River.
- Correlation between recorded rainfall and predicted wave heights at the beach (the break-out) so that reasonable assumptions of flood rainfall and corresponding water levels can be deduced.

The results of this analysis are presented in **Table 4.1** below, which contains the recommended tailwater conditions for Crooked River.

Table 4.1
Recommended Tailwater Boundary Conditions for Crooked River

	ENTRANCE OPEN	
1.	**Design water levels for the assessment of tailwater control on flooding from extreme precipitation**	
	Maximum tidal stage, storm surge plus greenhouse allowance	1.50 m AHD
	Nearshore wave setup	0.22 m
	Maximum still water level as tailwater control	**1.72 m AHD**
2.	**Design water levels for the assessment of tailwater control on flooding from extreme ocean water levels**	
	Maximum tidal stage, storm surge plus greenhouse allowance	1.70 m AHD
	Nearshore wave setup	0.75 m
	Maximum still water level as tailwater control	**2.45 m AHD**
	ENTRANCE CLOSED	
	For this condition, the enclosing beach berm is set at 2.1 m AHD (includes 0.2 m greenhouse rise). The ocean level for this condition is determined as follows:	
	Maximum tidal stage, storm surge plus greenhouse allowance	1.50 m AHD
	Nearshore wave setup	0.45 m
	Maximum ocean still water level	**1.95 m AHD**

5. HYDRAULIC MODELLING

5.1 HYDRAULIC MODEL

The MIKE-11 hydraulic model was adopted for this study. MIKE-11 is a hydraulic modelling package developed by the Danish Hydraulic Institute which has seen widespread application in Australia in recent years. The MIKE-11 computer model is capable of:

- analysing the flow distribution throughout a complex network of interconnected flow paths, each of which can have different hydraulic characteristics;
- adjustment so that it can analyse the effects of possible modifications which could influence flooding behaviour, such as construction of levees or channel modification;
- producing time series of flows, velocities and water surface elevations at nominated locations.

Although the computational procedure in the MIKE-11 model is based on one-dimensional flow equations, the model belongs to a family of "quasi two-dimensional" hydraulic models where a prior knowledge of the pattern of flood flows is required in order to set up the various channel and weir linkages which are used to compute the passage of a flood wave through the valley. This prior knowledge is obtained from inspection of aerial photographs and field inspection.

A topographic model of the channel and floodplain is set up using surveyed cross-sections at right angles to the direction of flow. The elevations of the tops of the channel banks are also required to enable computation of flow from the channel to the floodplain and vice versa. The model analyses the flow pattern, water levels and velocities throughout the study area at a point in time and takes account of differences in water level that cause water to move and the hydraulic resistance of the channel and floodplain. The model then steps forward in time to repeat the analysis a few minutes later. In this way, the model can take account of all the dynamic effects of flood waves from different parts of the catchment arriving at different times.

The principle features of MIKE-11:

- MIKE-11 is a “dynamic” model that takes account of the variation of flow and water levels over time.
- The model takes full account of the temporary storage of flood water within the river channel and flood storage areas such as Foys Swamp.
- The MIKE-11 model includes the capability to account for the progressive scouring of a channel through the beach and the effect of varying water levels within the estuary and the ocean as the scouring progresses.

The MIKE-11 model was set up to represent the conveyance of flood flows along the main flow paths as well as spillage from one flow path to another and the flow across the floodplain. This model has been configured to allow detailed assessment of the:

- flooding regime (flow depths and velocities);
- extent of flooding;
- effect of land use; and
- effects of construction or removal of banks.

More detail of the MIKE-11 modelling is contained in **Appendix D**.

5.2 MODEL STRUCTURE

A schematic layout of the MIKE-11 hydraulic model for Blue Angle Creek is shown on **Figure 5.1**. Cross-sections of the waterways and floodplain were obtained from field survey carried out specifically for this study. The choice of cross section locations depended on the need to represent significant features on the floodplain which influence hydraulic behaviour. The survey data was obtained in a digital format and then imported into the MIKE-11 model. Longitudinal sections of the top of the banks were also surveyed.

5.3 OVERVIEW OF FLOODING

Flooding in Foys Swamp is expected to be relatively frequent. The lowest parts of Foys Swamp are located adjacent to the Illawarra Railway Line. Ground levels in this area are in the order of RL 1.5 m AHD. At the mine site, ground levels are in the order of RL 2.0 m AHD and above.

Flooding in Foys Swamp is controlled by:

- local precipitation;
- prevailing ocean water levels;
- the state of opening of the Crooked River to the Pacific Ocean.

Precipitation is the main factor controlling flood levels within Foys Swamp while the prevailing ocean levels and entrance conditions are of lesser importance.

5.4 RESULTS

The impact of the existing mine and its proposed extension have been tested using the MIKE-11 hydrodynamic model described in **Appendix D**. The testing has been based on the following scenarios:

- 2002 mine layout and topography;
- 2005 mine layout and topography;
- 2005 proposed extension.

The “existing” and “proposed” conditions have been physically achieved in the MIKE-11 model by blocking the relevant parts of cross sections in the model. Thus, numerically, the area of the mine is effectively removed from the floodplain.

The MIKE-11 model results for the 20 and 100 year ARI events appear in **Table 5.1** below, and relate to the reference points shown on **Figure 3.1**. Note that the flood levels are given to 3 decimal place for **comparison purposes only**. The accuracy limits of the modelling would be to around 0.01 m.

The latest results for flood levels associated with the 2002 topography vary slightly from the results presented in the 2003 report. This is due to the inclusion of the additional cross sections in the model.

Table 5.1
Predicted Flood Levels

Scenario	Return Period (ARI years)	Flood Level at Reference Point (m AHD)			
		A	C	D	E
2002	100	2.631	2.629	2.628	2.615
	20	2.100	2.100	2.099	2.081
2005	100	2.644	2.643	2.641	2.624
(Existing)	20	2.106	2.105	2.104	2.082
Proposed	100	2.659	2.658	2.654	2.638
	20	2.111	2.111	2.103	2.086

The flood levels in Table 5.1 have been derived on the assumption of:

- 36 hour "critical" duration storm;
- tailwater set at RL 0.6 m AHD constant with time;
- Crooked River entrance as "open".

The modelling presented in **Appendix D** shows that this combination of boundary conditions produces a satisfactory estimate of the range of values that could be expected if the system is tested under differing tailwater and entrance conditions.

Table 5.2 below shows the affluxes resulting from the proposed sand mine extensions, compared with existing (2005) conditions.

Table 5.2
Affluxes Created – Proposed Extension of Sand Mine

Average Recurrence Interval (ARI)	2002 Flood Level (m AHD)	Fill Scenario	Afflux (mm)
100	2.631	2005 existing	13
		2005 extension	28
50	2.195	2005 existing	6
		2005 extension	12
20	2.100	2005 existing	6
		2005 extension	11
10	2.022	2005 existing	4
		2005 extension	8
5	1.960	2005 existing	3
		2005 extension	7

Review of the predicted flood levels in **Table 5.2** indicates the proposed mine extension, when compared to 2002 topography, will create a total afflux of between 7 – 28 mm. When compared to the existing 2005 topography, the afflux created is between 4 – 15 mm.

Affluxes of these orders are not considered significant given:

- the ground levels around Foys Swamp fall relatively steeply in the flood liable areas;
- there are no residential properties or buildings likely to be affected;
- the order of affluxes are very small and practically unmeasurable;
- the order of affluxes are very small, leading to a question of whether the affluxes are "real" or numerical in nature.

Table 5.3 below indicates the peak flow at the access road culvert (which is fitted with flap gates). Examination of **Table 5.3** shows that the transition from the 2002 topography to the current (2005) mine site and from the current mine site to the proposed mine site produces very little difference in the peak flows from Foys Swamp with the current conditions and the proposed extension to the sand mine. The results in **Table 5.3** have been produced based on the same assumptions as the results for **Table 5.1** and **Table 5.2** above (ie 36 hour "critical" duration storm; tailwater set at RL 0.6 m AHD constant with time and Crooked River entrance as "open").

Table 5.3
Design Peak Flows (m^3/s)

ARI	2002 topography	2005 existing	2005 extension
100 y	26.2	26.3	26.5
20 y	14.3	14.2	14.3

6. SUMMARY AND CONCLUSIONS

The conclusions that can be drawn from the investigations to date are as follows. Essentially, Foys Swamp and Blue Angle Creek above Reference Point E act as a large storage area during floods. The storage area drains to the Crooked River along Blue Angle Creek. The hydraulic model shows the proposed mine extension will create an afflux of 15 mm in the 100 year ARI flood compared to the existing (2005) mine conditions.

Affluxes of this order:

- can be viewed as more numerical than “practical”;
- will not impact on the properties surrounding Foys Swamp and Blue Angle Creek.

Given that Foys Swamp and Blue Angle Creek act as a flood storage area draining to the Crooked River by Blue Angle Creek, the total afflux caused by the proposed mine extension is not sufficient to cause changes in peak flows that affect other areas or changes in flow velocities to change current creek stability.

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CLEARY BROTHERS (BOMBO) PTY LTD
PROPOSED SAND MINE EXTENSION

LEGEND

Survey: PWD for new STP

Survey: Cleary Brothers (2003)

Survey: Cleary Brothers (2005)

Sand mine, pond area 2003

Sand mine, working area 2005

Sand mine, proposed extension

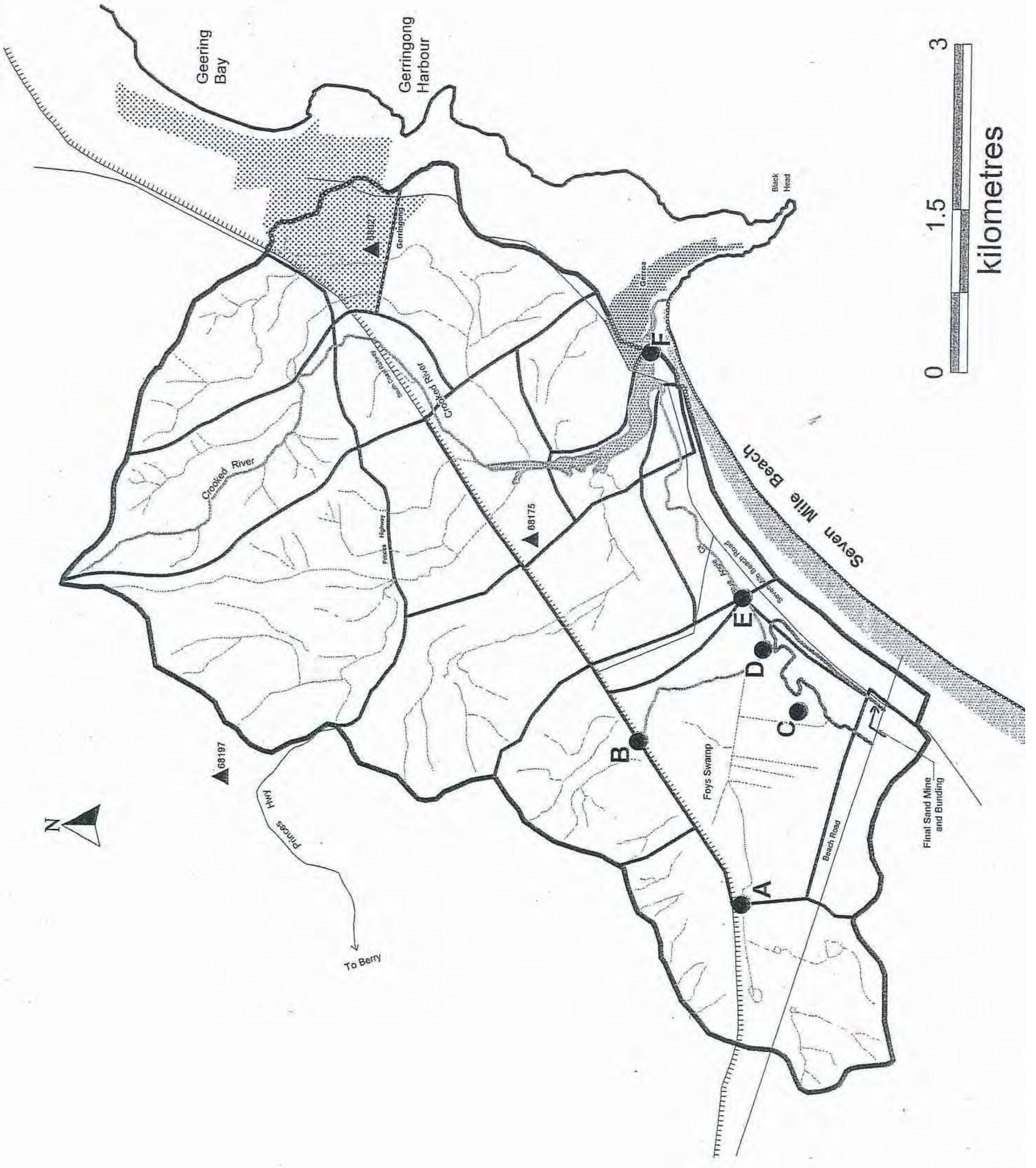
Reference Points (A)

0 1000 2000

Scale (metres)

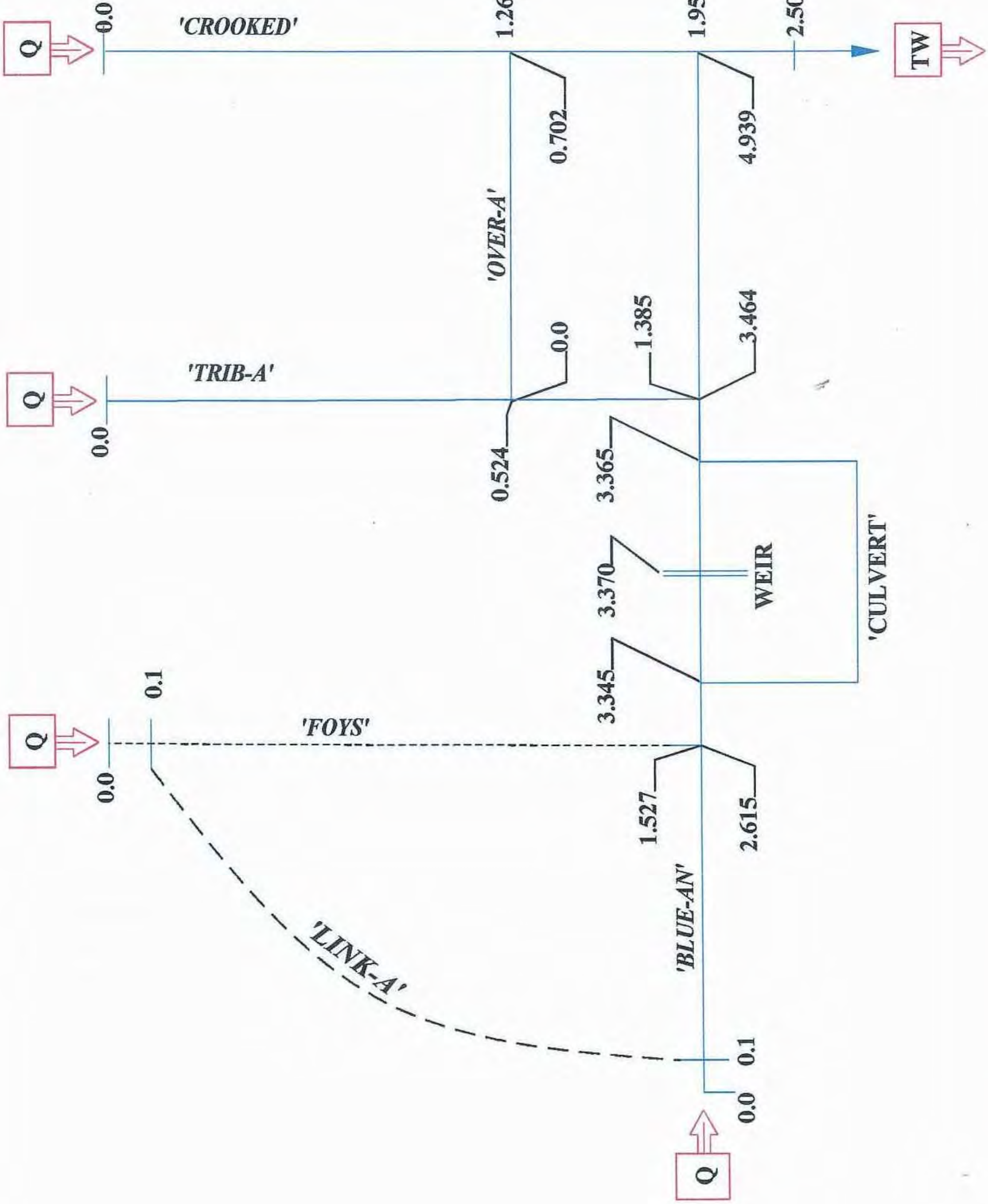
20 SEPT 2005
DISK REF: 05029
FIG REF: 05029_FIG1.1_V1

FIGURE 1.1
SITE PLAN



Legend		Project	Title
———	Sub-catchment Boundary	BLUE ANGLE CREEK FLOOD STUDY	RAFTS MODEL LAYOUT
●	Reference Point	**Details**	**Scale**
		Prepared by DH 12.11.02	1:50,000 — **Figure 3.1**

CLEARY BROTHERS (BOMBO) PTY LTD
PROPOSED SAND MINE EXTENSION



20 SEPT 2005
DISK REF: 05029_GERROA
FIG REF: 05029_Fig5.1_M11Schematic

FIGURE 5.1
MIKE-11 NETWORK

CLEARY BROS (BOMBO) PTY LTD

GERROA SAND MINE: FLOOD STUDY UPDATE

BLUE ANGLE CREEK FLOOD STUDY

Sand Mine Extension

APPENDICES

September 2005

Prepared by:

Evans and Peck Pty Ltd
20/390 Eastern Valley Way
East Roseville NSW 2069
Tel: (02) 9882 0500
Fax: (02) 9417 8850
Email: sperrens@evanspeck.com

Paterson Consultants
PO Box 596
Grafton NSW 2460
Tel: (02) 6643 1588
Fax: (02) 6642 7566
E-mail: patcon@nor.com.au

SMEC
PO Box 1052
North Sydney NSW 2059
Tel: (02) 9925 5555
Fax: (02) 9925 5566
E-mail: smecsyd@smec.com.au

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APPENDIX A

ANECDOTAL EVIDENCE RELATING TO CROOKED RIVER

(Provided by Mr Colin Stoddart, August 2002)

Anecdotal evidence regarding the opening of the Crooked River entrance was supplied by Mr Colin Stoddart, Town Planner and resident of Gerringong, based on ten years of records and observations.

> "The river entrance closes on a frequent basis, with at least four permanent closures and a number of closures of short durations during the 1990s. The closure of the entrance is encouraged by a number of events, including south-easterly seas and large movements of wind-blown sand off Seven Mile Beach. In some years substantial volumes of sand accumulates at the northern end of the beach. It is possible to view the sand being blown into the entrance. The threat of closure seems to be greatest during late autumn and early winter.
>
> The river at times has poor flushing characteristics. Over the decade I have observed periods of no tidal changes, with the river draining out to a trickle over a number of days. A change of tide usually occurs at a regular rate over 6 hours, whereas in Crooked River the run-out might extend beyond the 6 hour periods, with the run-in occurring over a very short periods.
>
> The river has at least three distinct characteristics. The entrance displays characteristics of a typical sandy entrance. Beyond Baileys Island the river becomes very shallow and has high levels of sediment. Wading/walking near water level at low tide is almost impossible. Access by canoe or boat is not possible during the lower half of the tide. The river begins to form a better channel beyond the electricity line which crosses the river to the west of Baileys Island. In the vicinity of Brian and Mina Sharpe's property, the river has a riparian edge with deep water, bank to bank. Much of the water stored in the river would accumulate at its western extremity. In periods of a partial closure and poor flushing I have concerns that sewage could flow into these deeper areas in the upper reaches. Its release into the sea could then take number of days or more.

River Openings and Closures (1990 – 2000)

1. River **opened** by Kiama Municipal Council late **November 1990**
   - river had been closed for some time
   - fish had gathered at the entrance trying to escape
2. River **opened late May 1995**
   - possibly closed over extended period over warmer months
   - poor water quality with water becoming turbid and algae present. Fish were in shallow water.
3. Possible **closure mid-April 1996 – late May 1996**
4. River **opened** about **12/7/1997** after closure of 6 – 8 weeks.
5. During **1998 and 1999** the river closed very briefly a number of times for a few days only. During autumn/winter 1998 the channel was very narrow and was almost closed (it closed briefly at least once). In late 1999 after exhibition of the EIS for the sewage treatment plant the river was observed to close briefly at least twice
6. Entrance **closed** again approximately **mid-June 2002**.

My recollection is that of the closures, three were opened after assistance from Kiama Council (mechanical opening). In addition, Kiama Council carried out major sand removal just to the west of the footbridge over an extended period in about 1996?

River Changes

The location of the river channel has changed east of Baileys Island over the decade. Up until 1995 the main channel was well formed, reasonably deep and hugging the northern bank. During the same period sediment loads west of the road bridge were much higher – it was much more difficult than at the present to wade near the main channel from the midpoint between the road and footbridges to the west.

Up until 1996 there was one main channel from east of the footbridge to just west of the road bridge towards Blue Angle Creek. During 1997 the channel east of the road bridge towards Blue Angle Creek began to change with a deep hole on south side of the river. After a large rain event in 1997 the main channel shifted away from the northern bank near the road bridge. The channel cut north to south under the bridge with a more direct connection to Blue Angle Creek. In 1997 Blue Angle Creek was quite clean with a good channel and deep holes. It fished very well.

Generally from 1997 – 1999 the depth of the channel east of the bridge has decreased considerably with more sand infilling. However, during 1999 deep hoes to rock formed at either side of the footbridge. Since the change of channel in 1997 connecting to Blue Angle Creek fishing in the creek deteriorated over the winters of 1998 and 1999. Visually the quality of the Blue Angle Creek environment has deteriorated. Over the last number of years (since 1998?) I reported two separate pollution events of Blue Angle Creek to Kiama Municipal Council and EPA officers.

It now appears that the channel is moving again with a deeper channel forming near the road bridge on the northern side.

These notes indicate that the river has poor flushing characteristics and that the lower reaches are constantly changing. The river entrance is unlikely to remain open. Not much is known about Crooked River, with no detailed studies to my knowledge.

At present (August 2002), the river is closed again. The entrance closed about mid-June 2002. There is more sand between the ocean and the river than I have seen before. Also, the channel east of the road bridge is very shallow."

APPENDIX B

RAFTS XP MODELLING

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B3.1 RAFTS Model Layout

B1. INTRODUCTION

The hydrologic computer modelling program RAFTS-XP (XP Software, 1996) was used to simulate the runoff characteristics of the Blue Angle Creek and Crooked River catchments. This model was utilised by GHD for previous studies and was adopted for this present study in the interests of consistency.

The hydrologic model was configured to be compatible with the hydraulic model, which required full flow hydrographs from catchments draining into Foys Swamp and Crooked River (rather than just the peak flow as used by GHD). The 20 and 100 year ARI events were modelled.

The RAFTS model represents the hydrologic behaviour of a catchment using a number of nodes and links. Nodes are used to represent sub-catchments, storage basins, reservoirs and confluences of creeks. The links normally represent flow paths between successive sub-catchments.

B2. MODEL SET-UP AND PARAMETERS

The RAFTS model for this present study includes all of the Crooked River and Blue Angle Creek catchments. Figure B3.1 shows the sub-catchment areas and the RAFTS model layout adopted for this study, together with the location of the Reference Points for generation of hydrographs for input to the hydraulic model.

B2.1 DEFINITION OF SUB-CATCHMENT AREAS

For modelling purposes sub-catchments were defined based on the topography of the catchment and locations where hydrographs were needed to establish boundary conditions for the hydraulic modelling. Topographic details were available from 1:25,000 topographic survey maps. This mapping was used to define the characteristics necessary for the RAFTS model for each of the sub-catchments, including sub-catchment area and main channel slope.

Estimates of sub-catchment roughness (PERN) were made based on the topographic mapping, knowledge of the catchment and indicative values obtained from the RAFTS Manual (XP Software, 1996), as listed below.

Table B2.1
Sub-Catchment Roughness Factors

Sub-Catchment Type	Sub-Catchment Roughness Factor
Impervious area	0.015
Urban pervious area	0.025
Rural pastures	0.05 – 0.07
Forested catchments	0.10

B2.2 DESIGN RAINFALL

Design storms for a range of frequencies up to the 100 year ARI were derived using principles given in Book II of Australian Rainfall and Runoff (1998).

B2.2.1 Rainfall Intensities

The relevant parameters from ARR Volume 2 (1987) was interpolated for the catchment. These parameters included design rainfall depths for 2 and 50 year ARI, and durations of 1, 12 and 72 hours, regional skewness and geographical factors. This enabled calculation of the rainfall intensity for the catchment based on procedures outlined in ARR99. Intensities were derived for frequencies of 2 to 100 year ARI and durations of 1 to 48 hours. Input of these parameters into the RAFTS model enables the model to calculate the design intensities, based on the methodology presented in ARR.

B2.2.2 Temporal Patterns

Temporal patterns are used in the conversion of a design rainfall depth with a specific ARI into a design flood of the same frequency. Patterns of average variability are assumed to provide the desired conversion. Temporal patterns for various zones in Australia are presented in Volume 2 of ARR and are in-built in the RAFTS model.

B2.2.3 Probable Maximum Precipitation

The Probable Maximum Precipitation (PMP) was estimated using the Generalised Short-Duration Method (GSDM) (Bulletin 53, BoM, 1994) as recommended in Book IV of ARR97.

On the eastern coast of Australia, the GSD Method can be applied to catchments up to 1,000 km^2 in area for a maximum duration storm of 6 hours. The catchment terrain category was assumed to be totally "rough", as it was located within 20 km from generally rough terrain.

The 6-hour total rainfall was calculated to be 670 mm, or an average intensity of 112 mm/h. The PMP intensity, together with the temporal pattern supplied in Bulletin 53, was entered into the RAFTS model to produce the PMF hydrographs for input to the hydraulic model.

B2.3 LOSSES

The initial and continuing loss method was used to calculate rainfall excess with the RAFTS model.

Walsh et al (1991) derived initial loss values using streamflow data from 22 rural gauged catchments and the design rainfall from ARR87 and recommended the values of initial loss shown in **Table B2.2** for non-linear modelling of catchments east of the Dividing Range.

Table B2.2
Recommended Initial Loss Values (ARR)

ARI (years)	Initial Loss (mm)
2	25
5	30
10	30
20	25
50	20
100	15

Book II in ARR98 recommends values of initial loss in the range 10 – 35 mm for catchments east of the Dividing Range. The GHD study adopted a value of 12 mm initial loss. A value of 10 mm was adopted for this current study, based on the recommendations in ARR and with reference to the PRM results (Section B4.1).

The recommended continuing loss adopted for design purposes is 2.5 mm/h

B2.4 CHANNEL ROUTING

Hydrographs were routed between nodes along the RAFTS links using lag times. Estimates of lag times were obtained using an approximate approach based on the Manning's equation.

B3. MODEL TUNING AND RESULTS

No observed flood hydrographs were available to calibrate the RAFTS model. "Tuning" of the model was therefore carried out by comparison of the results with a probabilistic rational method (PRM) analysis.

B3.1 PROBABILISTIC RATIONAL METHOD FLOWS

The PRM analysis was carried out for the catchment of the Crooked River just upstream of the confluence with Blue Angle Creek, a total of 20.5 km^2. The Blue Angle Creek catchment was excluded because the storage characteristics of Foys Creek were considered likely to produced anomalous results in the PRM. The analysis was carried out based on the approach outlined in ARR98 (Book IV).

The time of concentration for the area of interest was found to be 2.4 hours. The results of the analysis are contained in **Table B3.1**.

Table B3.1
Probabilistic Rational Method Results, Crooked River upstream of Blue Angle Creek

ARI (Years)	Intensity (mm/h)	Ffy	Cy	Q (m^3/s)	
				PRM	RAFTS
2	26.7	0.78	0.78	120	105
5	36.1	0.90	0.90	185	165
10	41.8	1.00	1.00	240	205
20	49.0	1.10	1.10	310	255
50	58.9	1.19	1.19	400	320
100	66.2	1.29	1.29	485	375

Note that the RAFTS model was operated for a 2 hour duration event, with an initial loss of 10 mm and a continuing loss of 2.5 mm/h.

B3.2 Comparison with GHD Results

Table B3.2 presents the peak flows derived from the present RAFTS analysis, together with those presented in the GHD report as well as the PRM analysis for the Crooked River upstream of the confluence with Blue Angle Creek. Table 3.2 also contains results for the same location but with Bx set at 2.3 for comparison with GHD's results. The RAFTS model results are for a 2 hour duration storm (20 year and 100 year ARI) and 6 hour storm for the PMF.

Table B3.2
Comparison of Design Flows, Crooked River upstream of Blue Angle Creek

Design Storm ARI (years)	Peak Discharge (m^3/s)			
	RAFTS model		GHD	PRM
	Bx = 1	Bx = 2.3	Bx = 2.3?	
20	255	110	135	310
100	380	165	190	485
PMF	840	695	640	-

It can be seen that the RAFTS model used for this study produces higher flow estimates than those adopted by GHD. The PRM analysis resulted in even higher flows than the RAFTS model.

It appears that the GHD model resulted in lesser flows due to the value adopted for Bx, the storage coefficient. Although not actually documented in their report, it appears that GHD have adopted a value for Bx of 2.3, while this current study has used a value of 1.0. Bx is the Storage Coefficient Multiplication Factor, which is used to modify the calculated Storage Time Delay Coefficient (B) and the default value for Bx is 1.0. Bx uniformly modifies all sub-catchment B values, thereby attenuating the peak discharge but not altering the total volume of the hydrograph generated. It can be seen that when the model is run with Bx set to 2.3, the results approach those achieved by GHD.

In addition, GHD used a slightly higher initial loss of 12 mm, compared with the 10 mm adopted for the present study.

It was considered that the adoption of the higher flows, generated using a Bx value of 1.0, is justified because:

- there is no justification to use a Bx value other than 1.0;
- alteration of Bx does not alter the volume of the hydrograph generated and this study is interested in identifying volumes to produce peak flood levels in Foys Swamp;
- the flows generated approach those produced using the PRM.

B4. DESIGN FLOWS

The RAFTS model was operated for storm durations of 2 hours to 48 hours, as shown in **Table** B4.1. The 2 hour event produced the peak flows from the catchment.

Table B4.1
RAFTS Flows (m^3/s), Crooked River Catchment Outlet

Duration (h)	20 year ARI	100 year ARI
2	**360**	**535**
4.5	330	470
6	360	490
9	345	390
12	300	405
18	245	335
24	290	380
36	260	345
48	265	355

However, subsequent hydraulic modelling showed that the 2 hour event was not the critical storm in terms of peak flood levels in Foys Swamp and at the development site. This was found to be the 36 hour event. A 6 hour duration storm was used for the PMF. Hydrographs were prepared for the 2, 5, 10, 20, 50, 100 year ARI 36 hour duration storm and the 6 hour PMF at all of the reference locations, for input to the MIKE-11 model.

APPENDIX C
OCEAN TAILWATER CONTROL

Prepared by SMEC Australia Pty Ltd

Paterson Consultants
on behalf of
Perrens Consulting

Crooked River Flood Study
Ocean Tail Water Control

Report

SMEC

Paterson Consultants
on behalf of
Perrens Consulting

Crooked River Flood Study
Ocean Tail Water Control

Report

Document Number 31333-018

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SMEC AUSTRALIA PTY LTD
ACN 065 475 149
Level 5, 118 Walker Street PO Box 1052 North Sydney 2060
Telephone (02) 9925 5555   Facsimile (02) 9925 5566

Prepared by:	Reviewed by:
Lex Nielsen Manager Coasts and Estuaries	Ian Varley Manager, Water Resources

FOREWORD

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This report was prepared by Messrs A F Nielsen and C A Adamantidis for the Client, Paterson Consultants on behalf of Perrens Consulting, for the purpose of determining appropriate tail water controls for a flood study of the Crooked River, Gerroa. These results are available for another purpose only with permission by the Client and SMEC Australia.

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1 INTRODUCTION

The Crooked River has its source on the southern slope of Currys Mountain a few kilometres inland from Gerringong. The river is narrow and around 8 km long, broadening in its lower reaches to form a sheltered lagoon estuary (Figure 1.1). The largest tributary, Blue Angle Creek, drains Foys Swamp to the south and joins Crooked River around 700 m upstream of the ocean entrance. The catchment of Crooked River has an area of around 32 km^2 and the waterway area of the estuary is around 0.4 km^2.

The river enters the ocean at the northern end of Seven Mile Beach, adjacent to Black Head and the small township of Gerroa (Plate 1.1). The entrance channel is trained on its northern side against Black Head and, generally, the inlet is open to the sea. However, the channel is closed, intermittently, by the transport and deposition of littoral drift from Seven Mile Beach. The estuary, therefore, may be categorised as one of the *Intermittently Closed or Open Lakes and Lagoons* (ICOLL).

The inlet faces SSE and Black Head protects the inlet from wave energy approaching from directions north of SSE. The offshore area at Black Head is characterised by bomboras (Figure 1.2), which provide further shelter to the inlet from wave energy. The beach and nearshore slope is relatively flat (Wright & Short, 1983) and has been surveyed by DPWS at around 1:55, which reflects the fine grain size of the beach sand (d_{50} = 0.20 mm – see Appendix B; C.E.R.C., 1984).

For any given rainfall event, the extent of flooding in the Crooked River catchment and its tributaries will be influenced by, *inter alia*, the state of the ocean inlet, whether open or closed, and the prevailing meteorological and tidal conditions. Whether the estuary entrance stays open or is closed will depend on the relative magnitudes of the littoral drift transport (and, hence, nearshore wave conditions) to the inlet, acting to close the entrance, and the ebb tidal discharge, which may be enhanced by precipitation, acting to keep the channel open.

Should the inlet be open to the sea, as portrayed in Plate 1.1, then ocean levels will provide a tailwater control on the discharge of estuary floodwaters. In the nearshore zone these levels will be influenced not only by the tidal stage and storm surge but also by nearshore wave and wind setup. Should the inlet be closed, the crest of the beach berm will present as a weir controlling the downstream water level for floodwater discharge. However, berm levels are low and wave action on elevated water levels will cause overtopping.

Previous flood studies for the Crooked River, which were carried out for the Gerringong Gerroa Sewerage Scheme project, were undertaken by Gutteridge Haskins & Davey (1999) and are reported in Devine & Chemke (1999). That work adopted what was identified as a conservative assumption of combining a 100 year ARI precipitation with a 100 year ARI sea level, the latter being one adopted from a study by Lawson & Treloar for the Shoalhaven River entrance.

This report documents a more detailed study that explores not only the coincidence of extreme precipitation and ocean wave and water level conditions but also water levels for various inlet configurations. The study establishes appropriate tailwater conditions for flood studies of the Crooked River.

2 METEOROLOGICAL CONDITIONS

2.1 Nearshore Oceanic Water Levels

During storms, the ocean water levels and those at the shoreline often are elevated above those of the normal tide. While these higher levels are infrequent and may last only for short periods, they allow larger waves to cross the offshore sandbars and reefs and break at higher levels on the beach. This may cause flooding of low-lying areas and may increase the tail water levels that control flood discharges of rivers.

The components of elevated oceanic storm water levels comprise astronomical, meteorological and global factors. All of the components do not act or occur, necessarily, independently of each other, but their coincidence and degree of interdependence, generally, is not well understood.

The storm surge and nearshore water level depends, primarily, on:

- the intensity, wind speed, scale, direction and speed of movement of the storm cell;
- the bathymetry of the coastal area including the presence or otherwise of offshore reefs and islands which condition wave transformation;
- the shape of the coastline including the topography of the nearshore areas;
- the prevailing barometric pressure (the inverse barometer effect); and
- the prevailing astronomical tide.

During extreme storms, the amount of storm surge resulting from barometric and wind setup on the NSW east coast, typically, can have a value of around 0.5 m. For example, during the severe storm of 24-25 May 1974, which is considered to have a recurrence interval of around 70 years (Lawson & Youll, 1977; Lord & Kulmar, 2000), the ocean water level measured at Fort Denison was 2.37 m on ISLW. This level was some 0.5 m above the predicted lunar tide of 1.9 m ISLW (Foster *et al.*, 1975). Of this anomaly, some 0.23 m can be attributed to barometric setup with the remainder being a water level setup resulting from onshore winds, which had an average speed of around 45 knots.

Modelling work undertaken by Lawson & Treloar (1987) for the determination of tailwater levels appropriate for flood studies on the Shoalhaven River indicated peak shoreline wind setup levels of around 0.2 m for surface wind speeds of around 60 knts.

Further elevation of the water level at the shoreline results from the breaking action of waves causing, what is termed, wave setup and wave runup. Wave setup may be perceived as the conversion of part of the wave's kinetic energy into potential energy as the wave shoals onto the beach. The amount of wave setup will depend on many factors including, among other things, the heights and periods of the waves, the nearshore bathymetry and the slope of the beach.

For the Shoalhaven River Flood Study, Lawson and Treloar (1987) computed a wave setup index on the closed entrance of 0.13 ($\times$ H′os – the deepwater unrefracted *significant* wave height), which was reduced for the condition where the entrance was open and waves propagated into the estuary. For the Shoalhaven River closed, a wave setup of some 1.5 m was computed from an 8 m deepwater *significant* wave height. This result was based on computed nearshore wave coefficients of around 1.5 (1.3 – 1.7), which resulted from wave focussing from refraction on the river entrance bar.

The energy of a wave is dissipated finally as the water runs up the beach or shoreline. Wave run-up is the vertical distance the wave will reach above the tide and storm surge level and can be several metres. At any particular site, wave run-up is very much a function of the foreshore profile, the surface roughness and other shoreline features on which the breaking waves impinge. Quantification of these factors relies on site-specific assessments.

In the longer term, there is a possibility that the meteorological conditions described above and sea levels may change as a result of an enhanced *Greenhouse Effect*. The term *Greenhouse Effect* is used to describe a postulated warming of the earth due to the accumulation in the atmosphere of certain gases, such as carbon dioxide, resulting from the burning of fossil fuels.

The most recent report of the Intergovernmental Panel on Climate Change (IPCC, 2001) indicated that, while temperatures may have risen in the atmosphere (+0.05° ± 0.10°C per decade since 1979), the data show that global average surface temperature has increased by +0.15° ± 0.05°C per decade since 1979. The warming has occurred, primarily, over the tropical and sub-tropical regions. However, the data on sea level rise is somewhat equivocal. While IPCC (2001) stated that global average sea level rose between 0.1 m and 0.2 m last century (average 1 mm/yr to 2 mm/yr), measured long-term trends in mean sea level (MSL) from Australian stations are less than those determined for most other locations (Cox and Horton, 1999). More recently, as measured over the 5 years to 1995, there has been a steep falling trend in MSL at Port Pierie (~24 mm/yr), Port Adelaide (~28 mm/yr), Fort Denison (~27 mm/yr) and Fremantle (~35 mm/yr). The MSL at Fort Denison in 1993 was about the lowest it has been since the mid 1940s and around the same as that recorded in the 1890s (Cox and Horton, 1999). The reason for this recent large rate of fall in Australian sea levels is not known, but a correlation with El Nino conditions through the Southern Oscillation Index has been suggested (Cox and Horton, 1999).

Nevertheless, IPCC (2001) advises that there may be a long term trend of rising sea, which has a projected range of sea level change from +0.09 m to +0.88 m between 1990 and 2100. The current mid-range scenario (most likely) IPCC sea level rise projections for 2050 and 2100 are 0.2 m to 0.5 m (+4 mm/yr to +5 mm/yr respectively). Global meteorological and oceanographic changes, such as the El Nino Southern Oscillation in the eastern southern Pacific Ocean, can cause medium-term variations in mean sea level of up to 0.1m.

2.2 Interdependence of Rainfall, Storm Surge and Heavy Seas

Both rainfall and heavy seas may affect flooding in the Crooked River estuary. The occurrences of heavy rainfalls and heavy seas are not, necessarily, statistically independent as both can be the result of the same weather system. Intense low pressure systems offshore that produce strong onshore winds, heavy seas and elevated nearshore water levels also are the mechanism driving large amounts of moist air onto the coast, resulting in heavy rainfall. However, heavy rainfall can occur without the occurrence of heavy seas.

To investigate the dependence or otherwise of extreme rainfalls and heavy seas, the storm data associated with the ten most severe rainfall events recorded at Gerringong since July 1895 were researched. The storm data, derived from Blain, Bremner & Williams (1985), Lawson & Treloar (1986) and data made available from the Manly Hydraulics laboratory, are presented in Table 2.1.

TABLE 2.1
STORM DATA FOR THE TEN HIGHEST-RANKED RAINFALL EVENTS RECORDED AT GERRINGONG

Rank	Date	3-Day Rainfall (mm)	Wind Speed (knts)	Wave Height (m)
1	21/11/1961	545	49	5
2	15/04/1934	429	35	4
3	18/05/1943	382	38	4 – 5
4	10/02/1958	355	-(1)	-
5	12/03/1975	343	-	-
6	08/07/1931	339	42	7
7	27/02/1919	335	-	-
8	30/04/1988	330	NA(2)	5
9	26/05/1900	316	34 – 40	3 – 5
10(3)	11/07/1904	315	34 – 40	5 – 6

Notes:
(1) - denotes Not Significant;
(2) NA denotes Not Available;
(3)"Nemesis" lost with all hands near Woollongong

The results revealed that six of these ten rainfall events were coincident with heavy seas (Hos ~ 5 m) and one of these events was associated with a very severe storm having an offshore significant wave height estimated at 7 m. The most severe rainfall event recorded was coincident with a severe storm event with peak *significant* wave height of around 5 m.

However, some of the most severe storms ever recorded in recent times on the south coast (1974 and 1978) were not associated with the highest ranked rainfall events. Table 2.2 presents rainfall associated with 2 extreme oceanic storms recorded on the south coast. These show that extreme oceanic storms are not necessarily coincident with extreme rainfall. Nevertheless, considerable rainfall was associated with these events.

TABLE 2.2
RAINFALL ASSOCIATED WITH
SELECTED EXTREME STORM EVENTS

Rank	Date	3-Day Rainfall (mm)	Wind Speed (knts)	Wave Height (m)
1	25-26/5/1974	70	60	9
2	31/5–2/6/1978	120	45	7

2.3 Design Meteorological Conditions

It would appear possible to combine the statistical properties of storm parameters with the results of surge modelling to determine the recurrence statistics of surge levels. However, there is some considerable difficulty in combining surge frequency statistics with tide height statistics. Methods based on the application of conditional probabilities have been applied but there are still difficulties in allowing for the variability of tidal amplitude, which is by far the greatest component of elevated water levels on the NSW coast, with wave setup, which introduces a further time-dependent variable. These complexities make the solution intractable.

The elevated ocean water levels, if presented as a stage hydrograph, would portray a tidal amplitude superimposed onto an elevated ocean level based on barometric, wind and wave setup that persisted for some 3 – 4 days. However, because the catchment is small, the flooding hydrographs would persist for only a few hours. Therefore, we see it appropriate to simply adopt the peak tailwater levels for flood assessment, rather than routing any elevated ocean water level hydrograph, which could extend for some 3 – 4 days, as a tailwater control for a flood model.

In view of the coincidence of severe storms with extreme precipitation (and *vice versa*) we recommend that flooding levels be assessed for the following peak tailwater conditions:

1) With the ocean entrance open:

- Flooding for extreme precipitation coinciding with a severe storm with offshore wave height of Hos = 5 m on a 1.5 m (AHD) ocean tide (includes storm surge and 0.2 m *Greenhouse* allowance) and with wave setup at the shoreline to be determined by site-specific analysis.

- Flooding with the ocean entrance open condition for a moderately heavy rainfall event (200 mm precipitation) coinciding with an extreme storm with wave height of Hos = 9 m on a 1.7 m (AHD) extreme ocean tide (includes storm surge and 0.2 m *Greenhouse* allowance) with wave setup at the shoreline to be determined by site-specific analysis.

2) With the ocean entrance closed:

- Flooding for extreme precipitation coinciding with appropriate berm levels coinciding with a severe storm with offshore wave height of Hos = 5 m on a 1.5 m (AHD) ocean tide (includes storm surge and 0.2 m *Greenhouse* allowance) and with wave setup at the shoreline to be determined by site-specific analysis.

3 NEARSHORE WAVE CONDITIONS

3.1 Introduction

The degree of wave exposure and, hence, water level setup at the ocean entrance to the Crooked River estuary has been investigated by undertaking a wave transformation study. The study comprised a 3-dimensional (2D-V) offshore to nearshore wave transformation analysis, using the SWAN program, to provide appropriate input data for a 2-dimensional (1D-V) nearshore wave transformation analysis, using the SBEACH program, to determine the wave and water level conditions at the ocean entrance of the Crooked River.

3.2 Offshore/Nearshore Wave Transformation

3.2.1 Modelling Algorithm

SWAN (acronym for **S**imulating **WA**ves **N**earshore – Cycle III version 40.11) is a numerical wave transformation program developed at the Delft University of Technology (Holthuijsen *et al.*, 2000). SWAN can be used to describe wave transformation in shallow water and to obtain realistic estimates of wave parameters in coastal areas, lakes and estuaries from given wind, bathymetric and current conditions. Of particular relevance to this study is that the wave-induced set-up of the mean sea surface is computed in SWAN.

SMEC has validated the SWAN numerical algorithms for the NSW coast *via* a comparison of numerical results with a comprehensive field data set obtained at Broken Bay, north of Sydney.

Details of the model and analysis for this project are in Appendix A.

3.2.2 Results

The analysis showed that the study site was most exposed to waves approaching from the ESE direction. For this direction, the wave height transformation coefficient as a result of the spectral wave refraction for peak wave periods of 10 s and 14 s for a range of offshore wave heights is depicted in Figure 3.1.

3.3 Nearshore Wave Transformation and Water Levels

3.3.1 Modelling Algorithm

The nearshore and shoreline wave setup was computed using the surf zone wave transformation program SBEACH (32 Version 2.0). SBEACH (**S**torm-Induced **BEA**ch **CH**ange 32) is an empirically based, two-dimensional, morphological, numerical model for simulating, *inter alia*, wave setup and runup water levels and wave-height variation across the surf zone.

The model is founded on extensive large wave tank and field data measurements and analysis (Larson *et al*., 1990; Rosati *et al*., 1993). The SBEACH algorithm has been validated for the Australian eastern seaboard for numerous Australian east coast sites (Carley, 1992; Carley *et al*., 1998).

The details of the model and analysis are in Appendix A.

3.3.2 Results

The variations in wave setup for both the entrance closed and open condition at Crooked River are presented in Figure 3.2 for a range of offshore *significant* wave heights with a peak wave period of 10 s approaching from ESE direction.

4 BEACH BERM LEVELS

4.1 Introduction

Entrance berms that close ICOLLs are depositional features resulting from the transport of littoral drift to the inlets under wave action. Should the estuary ebb tide or freshwater discharge be incapable of flushing out the littoral drift deposition then the inlet may close. The level to which the enclosing beach berm may grow is limited by the extent of incident wave runup (Gordon, 1990; Hanslow *et al*., 2000). The level of wave runup will be determined by the prevailing incident wave conditions and the characteristics (grain size) of the sediment, which determine the slope of the nearshore beach profile.

4.2 Field Data

The occurrence of entrance closure of the Crooked River was investigated by the inspection of available aerial photography as obtained by the Department of Land and Water Conservation (Table 4.1).

TABLE 4.1.
DATES AND OBSERVATIONS ON AERIAL PHOTOGRAPHY

Date	Observation of Photography
04-04-1949	Wide beach berm constraining narrow entrance channel along headland open to the sea. Wide Beach
29-12-1974	Narrow channel opening out to wide entrance over swash zone. (Small scale)
28-07-1977	Wide beach berm constraining narrow entrance channel along headland open to the sea. Narrow beach.
26-06-1979	Wide beach berm constraining narrow entrance channel along headland open to the sea.
09-01-1982	Relatively wide channel open to the sea. Wide beach. Transverse aeolian sand dunes
26-04-1984	Relatively wide and apparently deep channel open to the sea. Wide beach.
29-01-1993	Relatively wide and apparently deep channel open to the sea. Wide beach.
01-05-1993	Relatively wide and apparently deep channel open to the sea. Wide beach.
20-02-1999	Apparently deep but narrow channel open to the sea. Enclosing relatively high berm, above wetted line, with transverse aeolian sand dunes (see Plate 4.2).
15-04-2001	Narrow channel open to the sea across a low berm. Narrow beach.

As indicated in Table 4.1, of the ten photographs inspected from 4-4-1949 to 15-4-2001, the entrance channel appeared always to be open to the sea, at least on high tide. However, anecdotal evidence has indicated that the entrance has been closed from time to time.

From measurements made at a variety of NSW ICOLLs, Hanslow *et al.* (2000) postulated relationships between the levels attained by the enclosing berms and berm saddles (plugs in Gordon, 1990) and various characteristics of the entrances, such as sediment grain size, beach slope and entrance exposure. While the data from some fourteen ICOLLs portrayed a good deal of scatter, the data showed that berm saddle levels varied from around 1.9 m AHD to 3.0 m AHD and the adjacent beach berm levels varied from around 2.3 m to 3.6 m AHD. The lower values for berm and berm saddle (plug) levels were found to correlate with finer grained sediment, lower grade beach slopes and those inlets that were more sheltered from wave action. The levels attained by berm saddles (or plugs) was determined by the level of normal wave runup which, for Dee Why Lagoon, Gordon (1981; 1990) found to be around 1.6 – 2.0 m AHD and Hanslow *et al.* (2000) found to be around 2.2 m AHD.

The data presented in Hanslow *et al.* (2000) represented ICOLLs that were open only intermittently. However, as suggested in Hanslow *et al.* (2000), for ICOLLs that are, generally, open but may be closed only intermittently, then the beach and saddle berm levels may be lower. Photogrammetric plotting of the Arrawarra/Yarrawarra Creek entrance on Corindi Beach (DLWC, 1995), which is an ICOLL that, generally, is open, indicated that berm saddle levels were around 0.8 m AHD with adjacent beach berm levels reaching only 2.2 m AHD. It was noted that the Arrawarra/Yarrawarra Creek inlet was well sheltered from incident wave energy.

The Crooked River inlet lies on a section of coast that is sheltered from wave energy (Chapter 3 above; Appendix A), the beach berm comprises fine sand and the beachface has a nearshore beach slope of around 1:55. Based on the research presented in Hanslow *et al.* (2000), it may be expected that the berm saddle level at the Crooked River entrance would attain the levels commensurate only with the lowest values reported; that is, around 1.9 m AHD.

The available survey data from DPWS for the inlet open condition indicates an invert level for the entrance channel of around -1 m AHD (that is, around low tide level). The maximum berm level on the beach adjacent to the inlet channel, being indicated along the top of the swash zone on the beach, was measured consistently at 1.9 m AHD.

4.3 Analytical Assessment

When closed, as observed on a site inspection on 4th September 2002 (Plate 4.1), it would appear that the berm would not attain levels much above around 2 m AHD. To investigate further the potential level that the enclosing berm may reach, computations of irregular wave runup levels on smooth beaches were undertaken using the ACES (C.E.R.C., 1991) algorithms.

Computations were undertaken for a range of wave periods and for nearshore beach slopes of 1:50 and 1:60. The results, as shown in Figure 4.1, indicate that the maximum wave runup that can be expected at the site is around 0.6 – 0.8 H'os (the unrefracted deepwater *significant* wave height).

The annual average deepwater *significant* wave height is 1.6 m (Lord & Kulmar, 2000). From the wave transformation analysis as documented in Chapter 3, the nearshore wave height coefficient for the site is 0.40. This gives an annual average unrefracted deepwater *significant* wave height of 0.64 m for the site. For this wave height, the expected wave runup at the site would be 0.5 m, giving a berm level of 1.5 m AHD.

4.4 Adopted Berm Level

The observed berm level at 1.9 m AHD is above that determined, analytically, to be 1.5 m from the annual average wave height. There is no reason for the berm level to reflect the wave runup from the annual average wave height over any other wave height parameter. However, we are not aware of any further research in this area.

Therefore, we have adopted the level of 1.9 m AHD as the appropriate berm closure level for the entrance closed condition of the flood study.

5 RECOMMENDED TAILWATER CONTROLS

For the Design Meteorological Conditions as set out in 2.3 above, the recommended tailwater controls are set out below:

5.1 Entrance Open

5.1.1 Design water levels for the assessment of tailwater control on flooding from extreme precipitation:

Maximum tidal stage, storm surge plus *Greenhouse* allowance:	1.50 m AHD
Nearshore wave setup:	0.22 m
Maximum still water level as tailwater control:	**1.72 m AHD**

5.1.2 Design water levels for the assessment of tailwater control on flooding from extreme ocean water levels:

Maximum tidal stage, storm surge plus *Greenhouse* allowance:	1.70 m AHD
Nearshore wave setup:	0.75 m
Maximum still water level as tailwater control:	**2.45 m AHD**

5.2 Entrance Closed

For this condition, the enclosing beach berm is set at 2.1 m AHD (includes 0.2 m *Greenhouse* rise). The ocean level for this condition is determined as follows:

Maximum tidal stage, storm surge plus *Greenhouse* allowance:	1.50 m AHD
Nearshore wave setup:	0.45 m
Maximum ocean still water level:	**1.95 m AHD**

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Figures

Figure 1.1 – Plan of Crooked River Estuary

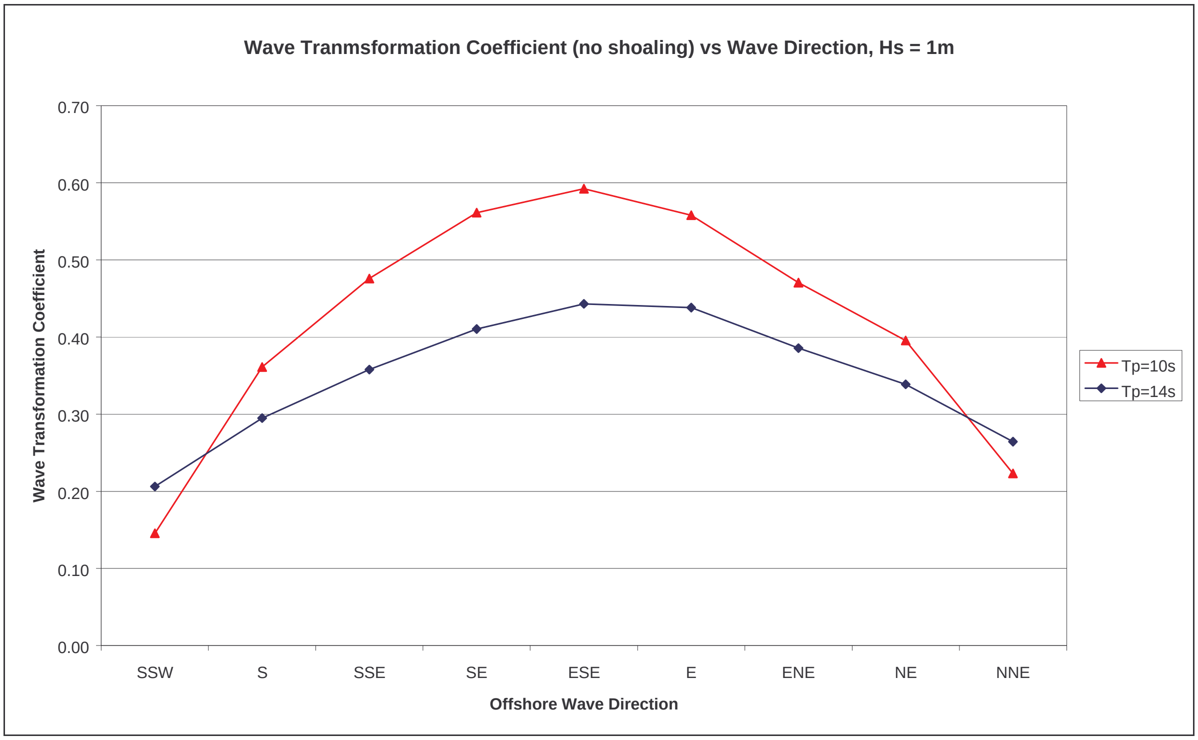

Figure 3.1 – SWAN Wave Transformation Coefficient vs Offshore Wave Direction

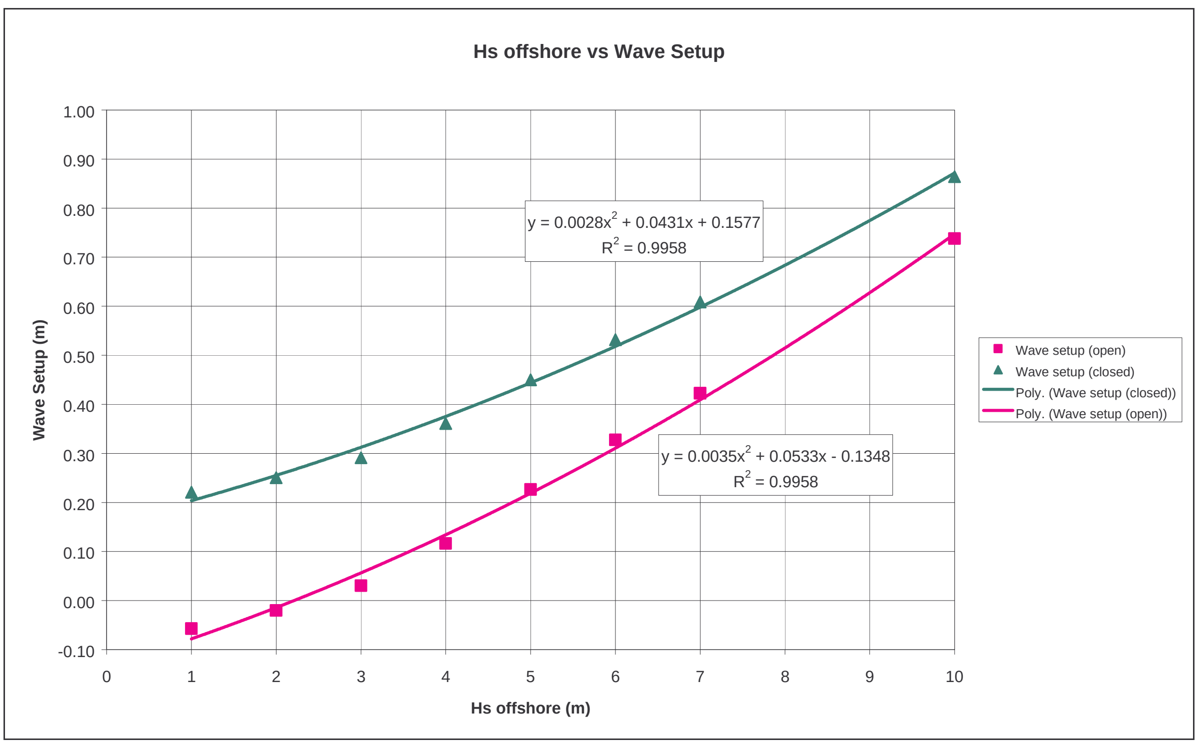

Figure 3.2 –Wave Setup vs H_s (offshore) at Crooked River Entrance for Tp=10s

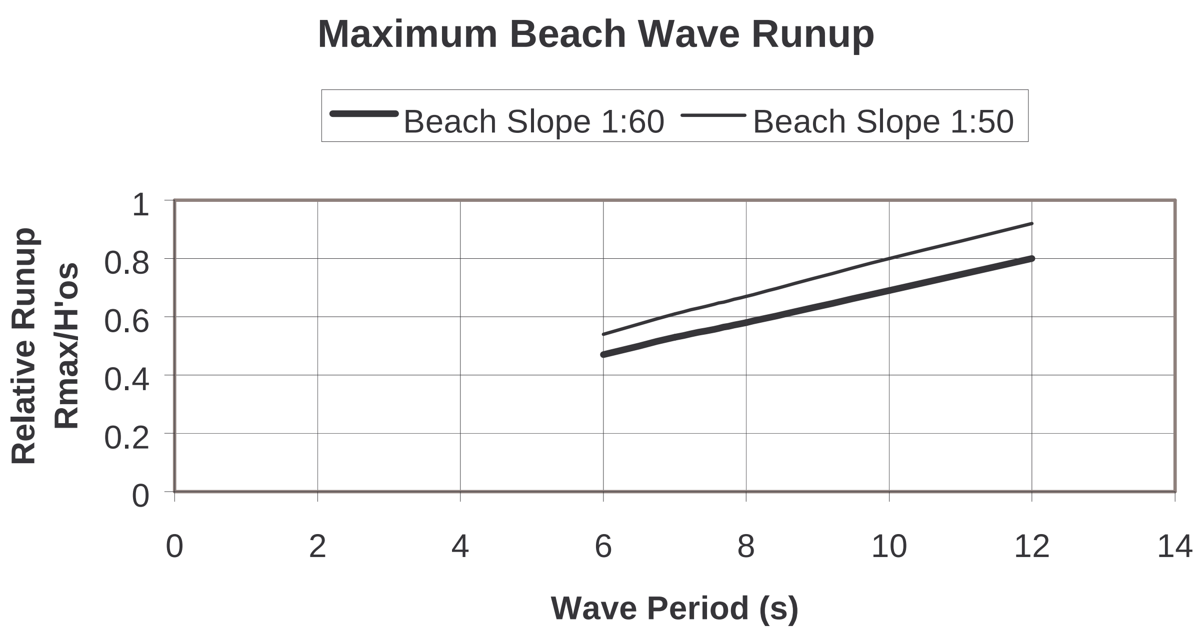

Figure 4.1 – Computed Range of Relative Levels of Maximum Irregular Wave Runup for Various Peak Periods on Smooth Beaches of Various Slopes

Plate 1.1 – Ocean Inlet of the Crooked River Estuary

Plate 4.1 – Crooked River Ocean Entrance 04-09-2002

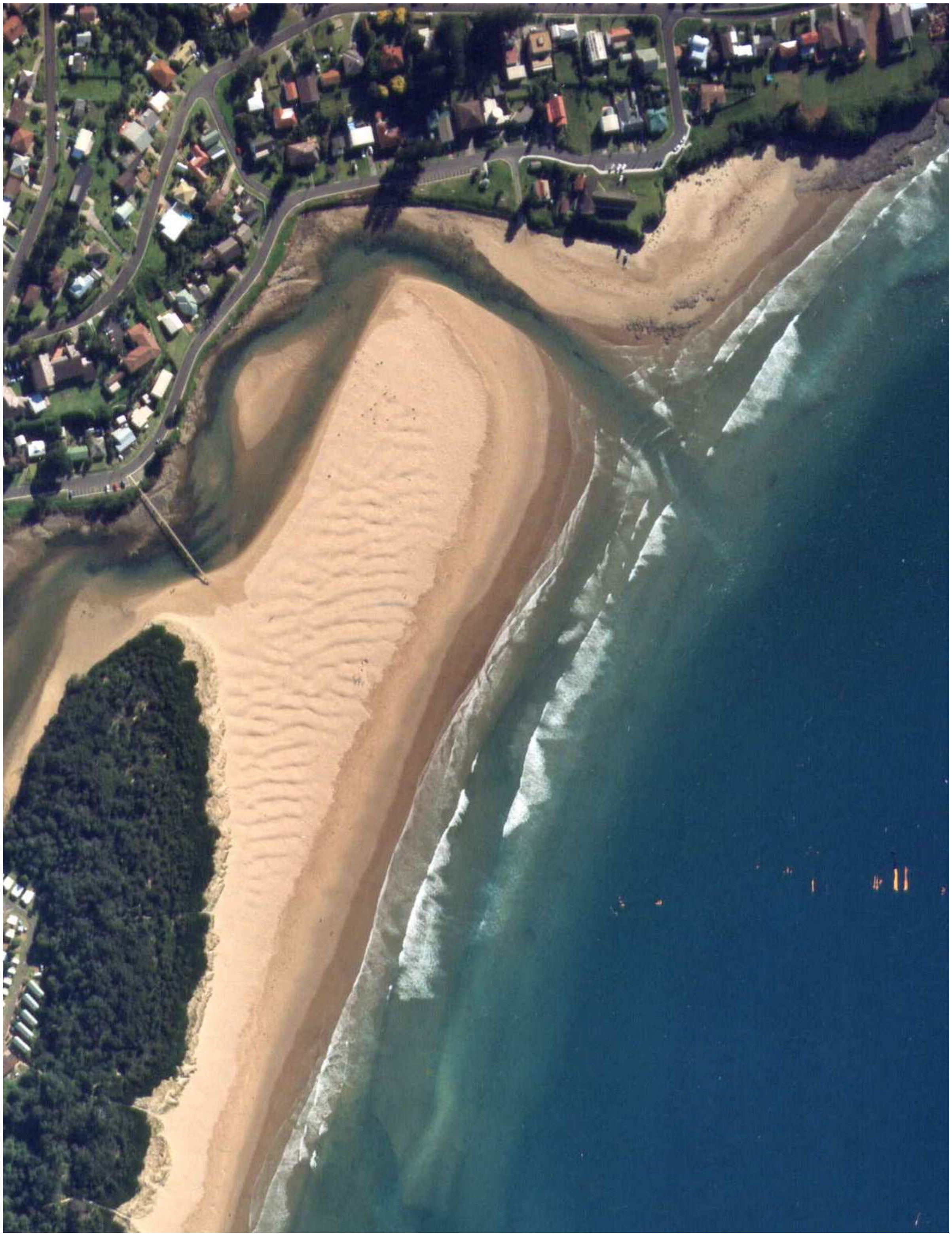

Plate 4-2 – Crooked River Ocean Entrance 20-02-1999.
Photograph shows an enclosing berm being relatively high above the wetted line and overlain with transverse aeolian sand dunes

APPENDIX A

Wave Transformation Modelling

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1 INTRODUCTION

The wave conditions at the ocean entrance to Crooked River were assessed by undertaking a two-stage wave transformation numerical modelling method:

1). wave transformation from deepwater, through transitional waters, to the study area; and

2). nearshore wave transformation including wave shoaling and wave setup.

The transformation of deepwater waves to the study site was undertaken using the SWAN numerical algorithms. This stage provided a quantified description of the modification that occurs to waves approaching the study site from deep water resulting from irregular bathymetry. It comprises, essentially, the effects of wave refraction.

The final stage utilised the SBEACH numerical algorithms, providing a detailed description at a very fine resolution of the wave shoaling processes across the nearshore beach zone (surf zone) producing shoreline wave setup and runup.

The numerical algorithms adopted for both stages were applied to a spectral schematisation for irregular waves (rather than to a regular [i.e., monochromatic] wave condition).

2 OFFSHORE TO NEARSHORE WAVE TRANSFORMATION

2.1 Introduction

SWAN (acronym for **S**imulating **WA**ves **N**earshore – Cycle III version 40.11) is a numerical wave transformation program developed at the Delft University of Technology (Holthuijsen *et al.*, 2000). SWAN can be used to describe wave transformation in shallow water and to obtain realistic estimates of wave parameters in coastal areas, lakes and estuaries from given wind, bathymetric and current conditions.

SWAN is based on the wave action balance equation (or energy balance in the absence of currents) with sources and sinks. The background to SWAN is provided in Young (1999) and Booij *et al*., (1999).

The following wave propagation processes are represented in SWAN:

- rectilinear propagation through geographic space;
- refraction due to spatial variations in bottom topography and current;
- shoaling due to spatial variations in bottom topography and current;
- blocking and reflections by opposing currents;
- transmission through, blockage by or reflection against obstacles.

The following wave generation and dissipation processes are represented in SWAN:

- generation by wind;
- dissipation by white-capping;
- dissipation by depth-induced wave breaking;
- dissipation by bottom friction;
- wave-wave interactions (quadruplets and triads);
- obstacles.

Of particular relevance to this study is that the wave-induced set-up of the mean sea surface is computed in SWAN. In (geographic) 1D cases the computations are based on exact equations. In 2D cases, the computations are based on approximate equations as the effects of wave-induced currents are ignored (in 1D cases they do not exist).

Diffraction is not modelled in SWAN, so SWAN can not be used in areas where variations in wave height are large within a horizontal scale of a few wavelengths. Because of this, the wave field computed by SWAN will, generally, not be accurate in the immediate vicinity of obstacles and certainly not within harbours.

SWAN does not calculate wave-induced currents. If relevant, such currents can be provided as input to SWAN (*e.g.* from a hydro-dynamical model, which can be driven by waves from SWAN in an iterative procedure).

2.2 Modelling Method

2.2.1 Objective

The objective of the SWAN wave transformation modelling exercise was to derive a set of nearshore wave coefficients for the range of offshore wave directions and periods comprising the long term wave climate to establish suitable input boundary conditions for the SBEACH surfzone wave transformation model. The SBEACH model was utilised to describe in detail the surfzone wave transformation processes for the determination of nearshore wave setup water levels and, hence, tailwater control levels for the flood model.

2.2.2 Bathymetric and Wave Data

Bathymeytric data for the model comprised:

- digitised soundings on a 1 km grid as provided by Geoscience Australia (Petkovic & Buchanan, 2002); and

- detailed soundings of the nearshore area of the Crooked River entrance, undertaken by Department of Land and Water Conservation (DLaWC) on behalf of NSW Department of Public Works and Services (DPWS), at a an approximate grid spacing of 20 m.

Long term wave statistics were derived from a directional Waverider buoy operated by the Manly Hydraulics Laboratory, DPWS offshore of Sydney as published in Lord and Kulmar (2000).

2.2.3 Model Schematisation

The domain of the wave transformation model extended from Stanwell Park in the north to south of St Georges Head, Jervis Bay, extending some 50 km offshore into water depths in excess of 100 m (Figure 2.1). This region was schematised onto a 1km square grid from data derived from the soundings on the 1 km grid.

A 400 m nested grid, extending from the Beecroft Peninsula to Kiama out to 100 m depth, provided a more detailed schematisation of the study region (Figure 2.2). Data for this grid was derived from the 1 km grid as provided by Geoscience Australia supplemented with detail from the *Aus. 808* Admiralty Chart *Jervis Bay to Port Jackson*, particularly for the area surrounding Sir John Young Banks.

Details in the nearshore area of interest were schematised on a 10 m grid based on the recent detailed DLaWC/DPWS survey and is depicted in Figure 2.3.

Figure 2.4 shows the nearshore bathymetry as seen by the SWAN program.

2.2.4 Model Validation

Data were not available for a site-specific validation of the wave transformation model. However, SMEC has validated successfully the SWAN numerical algorithms for the NSW coast *via* a comparison of numerical results with a comprehensive field data set obtained at Broken Bay, north of Sydney.

3 NEARSHORE WAVE TRANSFORMATION

3.1 Introduction

Numerical cross-shore wave transformation modelling has been undertaken to assess in detail the water level variations in the nearshore zone applicable to setting tailwater levels for the flood study.

The numerical model chosen for this assessment was SBEACH (32 Version 2.0). SBEACH (**S**torm-Induced **BEA**ch **CH**ange 32) is an empirically based, two-dimensional, morphological, numerical model for simulating, *inter alia*, wave setup and runup water levels and wave-height variation across the surf zone. The model is founded on extensive large wave tank and field data measurements and analysis (Larson *et al*., 1990; Rosati *et al*., 1993). The model accepts as data:

- surveyed beach profiles;
- time-varying water levels;
- regular or irregular wave heights and periods;
- wave angles;
- wind speeds and wind directions; and
- an arbitrary grain size in the fine-to-medium sand range.

3.2 Modelling Method

3.2.1 Modelling Approach

The approach to the SBEACH modelling has been to schematise the beach based on the detailed survey data and to assess the water level variations across the surf zone for the entrance open and closed conditions. This was undertaken to examine the impact of very severe storms, such as those of 1974, as well as for lesser events.

3.2.2 Model Schematisation

The beach profile used in SBEACH was schematised based on:

- digitised soundings on a 1 km grid as provided by Geoscience Australia; and

- detailed soundings of the nearshore area of the Crooked River entrance, undertaken by Department of Land and Water Conservation (DLaWC) on behalf of NSW Department of Public Works and Services (DPWS), at a an approximate grid spacing of 20 m.

The incident refracted wave path was determined using the results of the SWAN wave transformation model, and the beach profile along this wave path was used as input to the SBEACH model. The profile used in the SBEACH modelling is shown in Figure 4.3. Grain size data are from Appendix B.

The model was developed on the very severe storm of May 1974. As deepwater data were not available for the 1974 event, a time history for the event was synthesised from various sources after Nielsen *et al.* (1993). For this storm, the deepwater wave height peaked at Hs = 10 m. An Hs of 9 m was given a duration 10 hrs, that of 8 m was given 20 hrs and that of 7 m some 40 hrs. The event included a storm surge of some 0.5 m. The wave period adopted for the 1974 events varied from 6s at the start of the event, increasing to 12 s after 72 hours and remaining constant thereafter. According to Lord and Kulmar (2000), this synthetic storm would have a return interval in excess of 100 years.

The nearshore, wave-height history for this storm at some 30 m water depth was synthesised from the deepwater data by the application of a range of wave height coefficients, as determined by the SWAN transformation modelling. The purpose of this exercise was to determine a relationship between unrefracted offshore wave height and wave setup, as is discussed in Section 3.

3.2.3 Model Validation

There are no site data with which to validate the program. However, the SBEACH algorithms have been validated for the Australian eastern seaboard at numerous sites (Carley, 1992; Carley *et al*., 1998).

4 MODELLING RESULTS

4.1 Introduction

The SWAN model was run for a range of offshore wave heights, periods and directions, in order to determine the relationship between offshore wave parameters and the nearshore wave transformation in the vicinity of the Crooked River entrance.

4.2 Preliminary SWAN Modelling

Preliminary SWAN modelling was carried out, in order to determine the offshore wave conditions which would lead to the highest wave climate at the Crooked River Entrance. A point on the SWAN grid opposite the river entrance and in 4.3 m water depth was chosen for the purpose of reporting the model results (ISG coordinates 300000E, 6149500N).

For the preliminary modelling, an offshore wave height of 1m and offshore wave directions varying from SSW to NNE were used, to determine the direction at which maximum wave energy is refracted into the area opposite the river entrance. The 1m wave height was chosen to minimise the effect of shoaling on the wave transformation coefficient. Figure 4.1 shows the wave transformation coefficients for 1m offshore waves from a range of directions, for peak wave periods of 10s and 14s. It can be seen from this figure that maximum wave energy is refracted into the region of interest for an offshore wave direction of ESE.

4.3 Refracted Wave Path

Figure 4.2 is a vector diagram indicating the refracted wave paths and wave transformation coefficients at various locations surrounding the river entrance and along Seven Mile Beach, for an offshore wave height of 1m and wave direction of ESE. The area at the northern end of the beach at the location of the Crooked River Entrance is flanked by an offshore reef extending south from Black Point, which in some places is only 2m below the water surface. This reef results in strong refraction effects and has a profound influence on the wave climate at the river entrance. A wave path was drawn based on the refracted wave vectors, in order to determine a bathymetric profile for the wave setup calculations in SBEACH. This wave approach path is indicated in Figure 4.2, and the bathymetric profile along this wave approach path is shown in Figure 4.3.

4.4 Wave Transformation due to Refraction

The wave setup at the river entrance will vary based on the offshore wave height and the resulting nearshore wave conditions. To determine the wave setup at the entrance, the SBEACH program was used; this program does not take into account wave refraction. SWAN calculates a nearshore wave transformation coefficient based on the combined effects of refraction and wave shoaling; as such, the wave transformation coefficients due to refraction were determined using SWAN, and by removing the component of the wave transformation due to shoaling.

In order to determine the wave transformation coefficients due to shoaling, maximum wave heights from the SBEACH modelling were extracted at a chainage along the SBEACH profile where the depth was 4.3 m below still water level. Waves were transformed in SBEACH from deep water to a depth of 4.3 m, equivalent to the depth at which the analysis was carried out in front of the river entrance. An ***unrefracted wave***

transformation coefficient was determined for a range of offshore significant wave heights - this relationship is shown in Figure 4.4. As can be seen in this figure, the relationship between the unrefracted wave transformation coefficient and offshore significant wave height does not depend on whether the river entrance is open or closed.

SWAN was then run for a range of offshore wave heights, a peak wave period T_p = 10s, and an offshore direction of ESE. These SWAN runs provided wave transformation coefficients, from which Figure 4.4 was used to remove the component of the wave transformation due to wave shoaling and dissipation effects. The resulting relationship between offshore wave height and SWAN wave transformation coefficient is illustrated in Figure 4.5.

Generalised SWAN results for transformed significant wave height and transformed mean offshore wave period, vs offshore significant wave height for a peak wave period of T_p=10s and direction of ESE are provided in Figure 4.6.

4.5 SBEACH Wave Setup Calculations

The SBEACH model was then run for a range of wave heights using the configuration of the 1974 storm, for conditions when the lagoon entrance is both open and closed. SBEACH calculated a wave setup at the river entrance based on a range of wave heights, and a relationship between wave height and wave setup was obtained for the two conditions of an open and closed entrance. This relationship is depicted in Figure 4.7.

4.6 Ocean Tail Water Control Results

Figure 4.8 combines the results from Figures 4.5 and 4.7, and provides a design tool for obtaining the wave setup at the river entrance under a range of offshore wave conditions. The relationship shown in figure 4.8 between offshore significant wave height and wave setup at the entrance to Crooked River was calculated using relationships derived from Figures 4.4, 4.5 and 4.7.

For example, from Figure 4.8, an offshore significant wave height of 5 m results in a wave setup of 0.22 m at the river entrance, when the entrance is open. When the entrance is closed, the wave setup is approximately 0.45 m. For an offshore significant wave height of 10 m, the wave setup is approximately 0.75 m for an open lagoon entrance and 0.85 m for a closed lagoon entrance.

The wave setup, combined with tidal conditions, barometric setup and sea level rise due to climate change, provides the ocean tailwater condition for the flood model at the river entrance.

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Figures

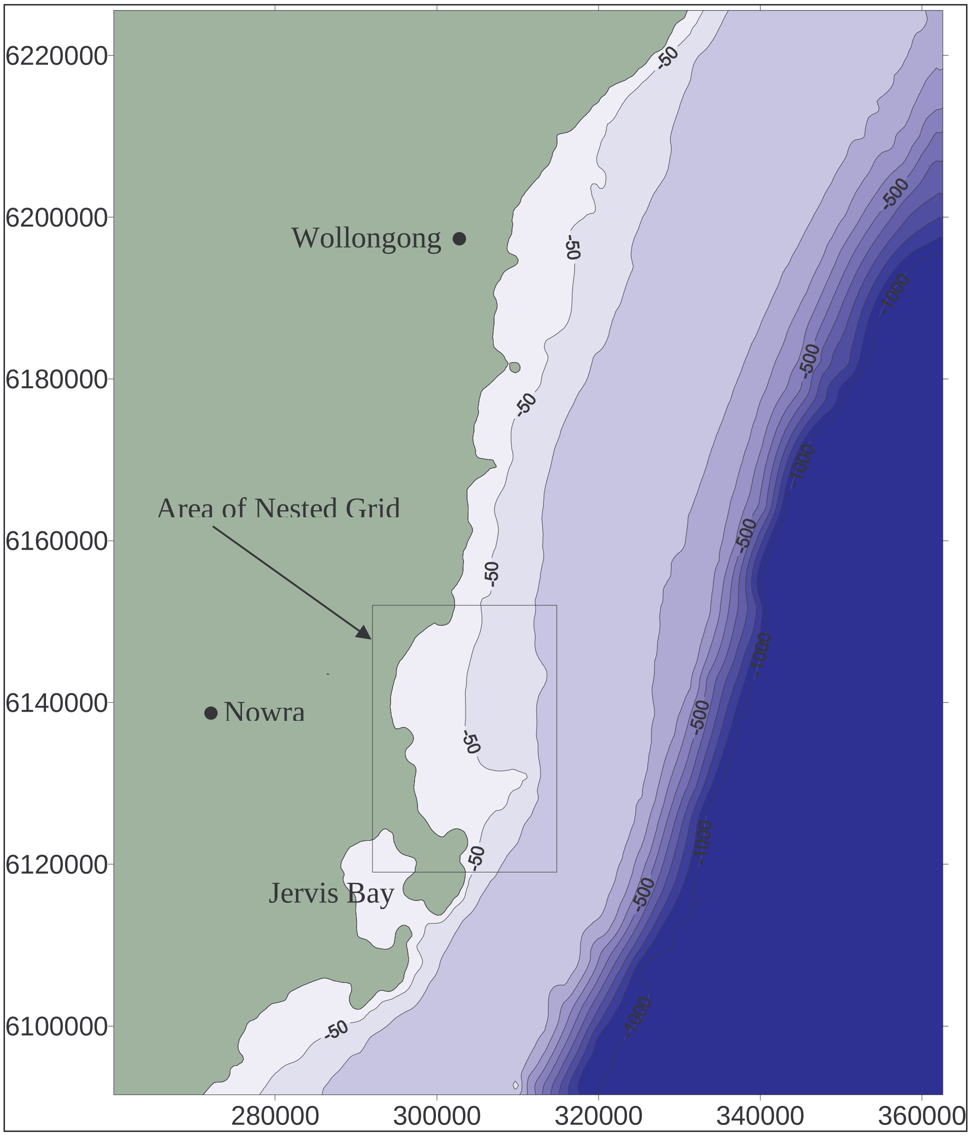


Figure 2.1. Wave Transformation Model Domain

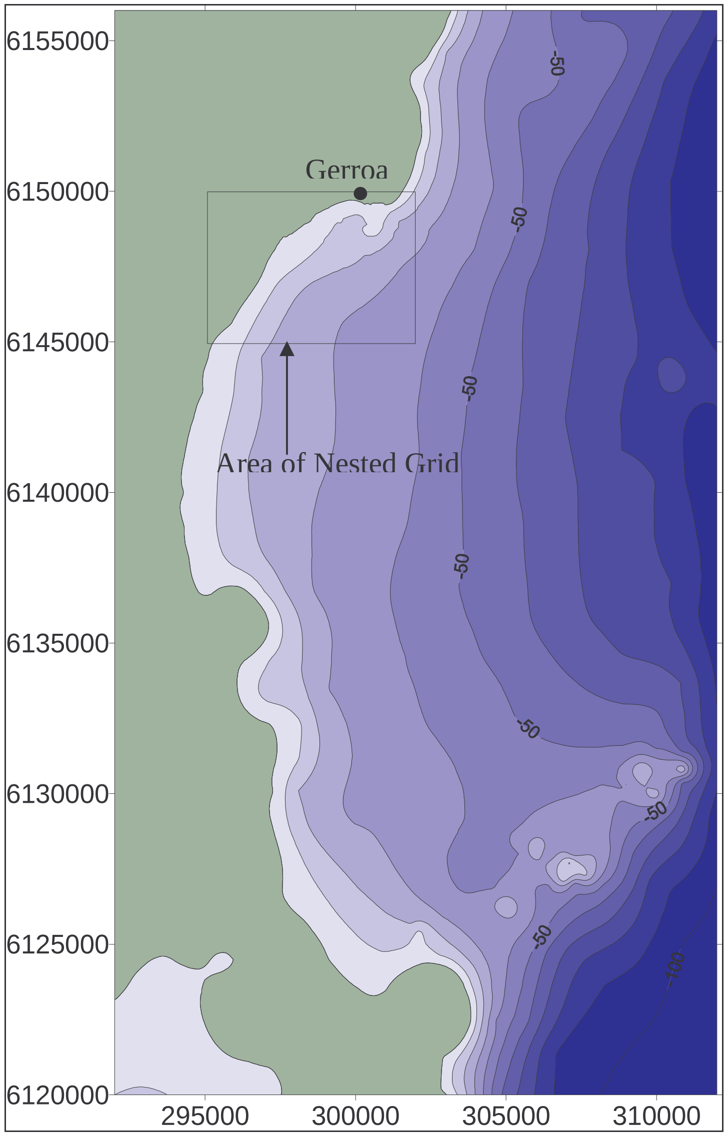


Figure 2.2 - Second Level Nested Grid Area

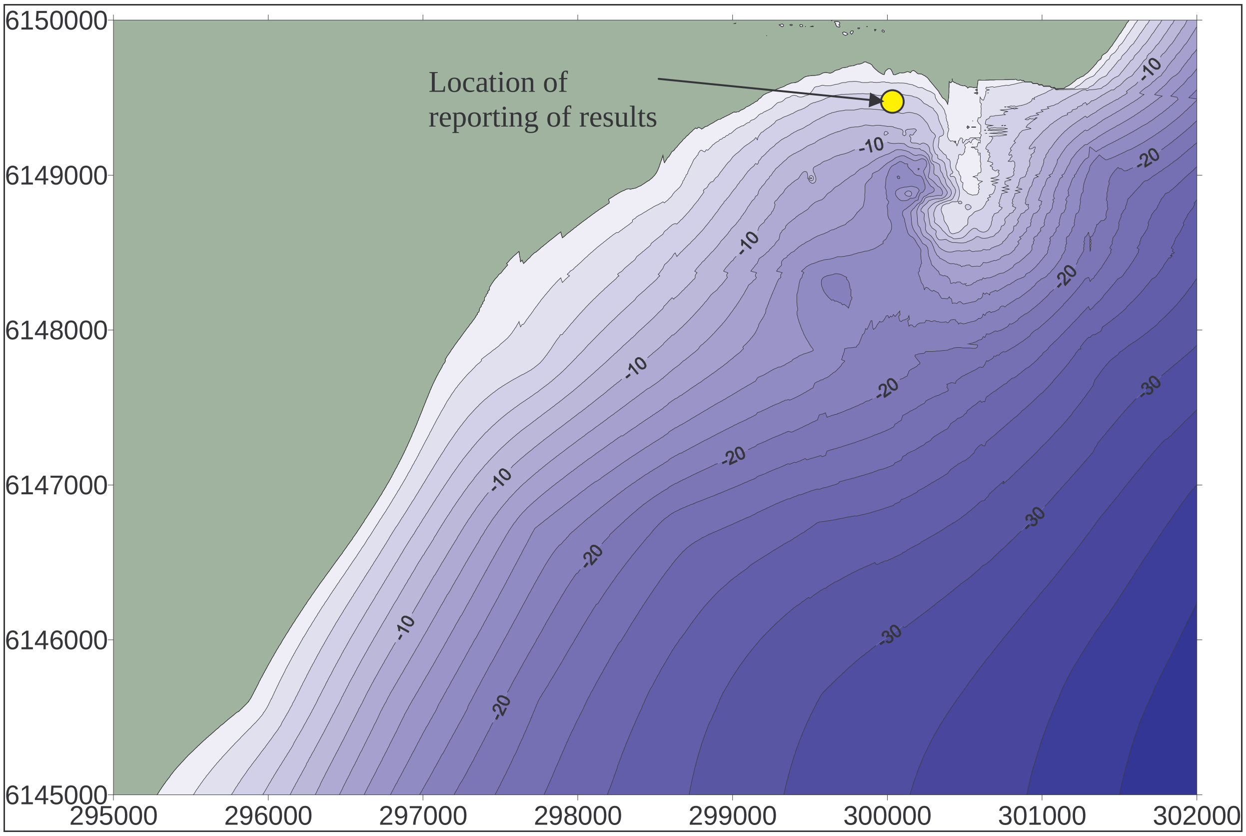


Figure 2.3 - Detailed Nested Grid Area

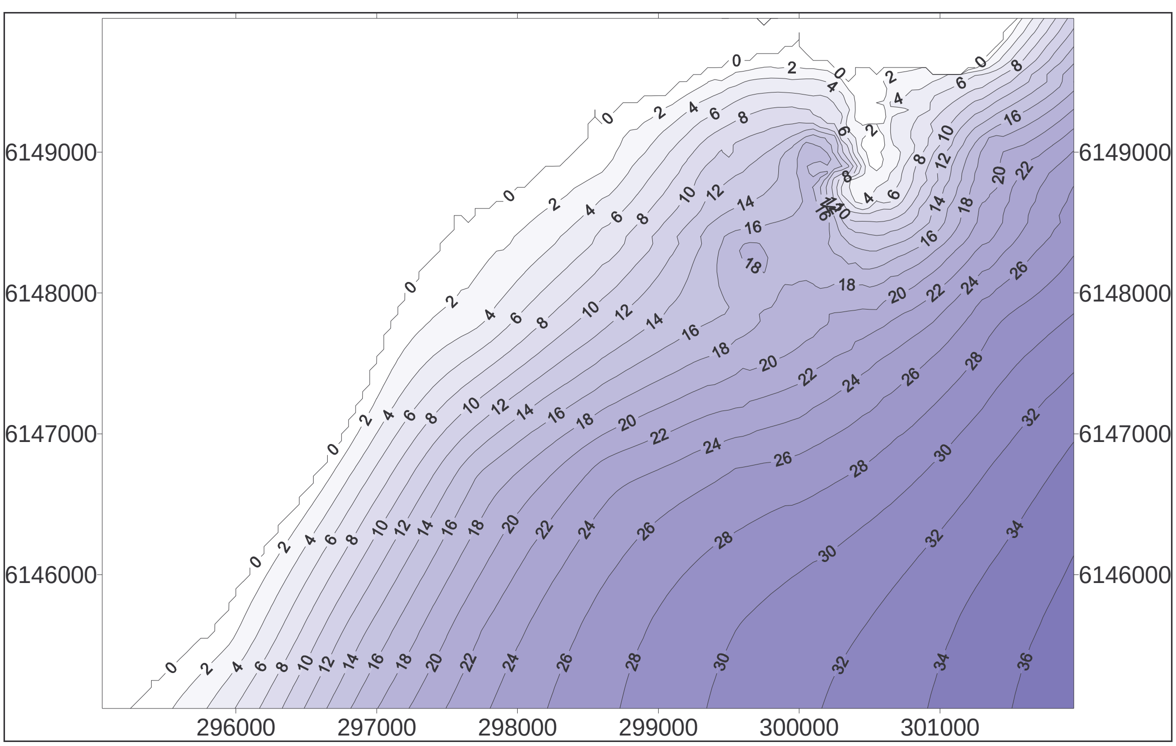


Figure 2.4 - Nearshore bathymetry as seen by SWAN program

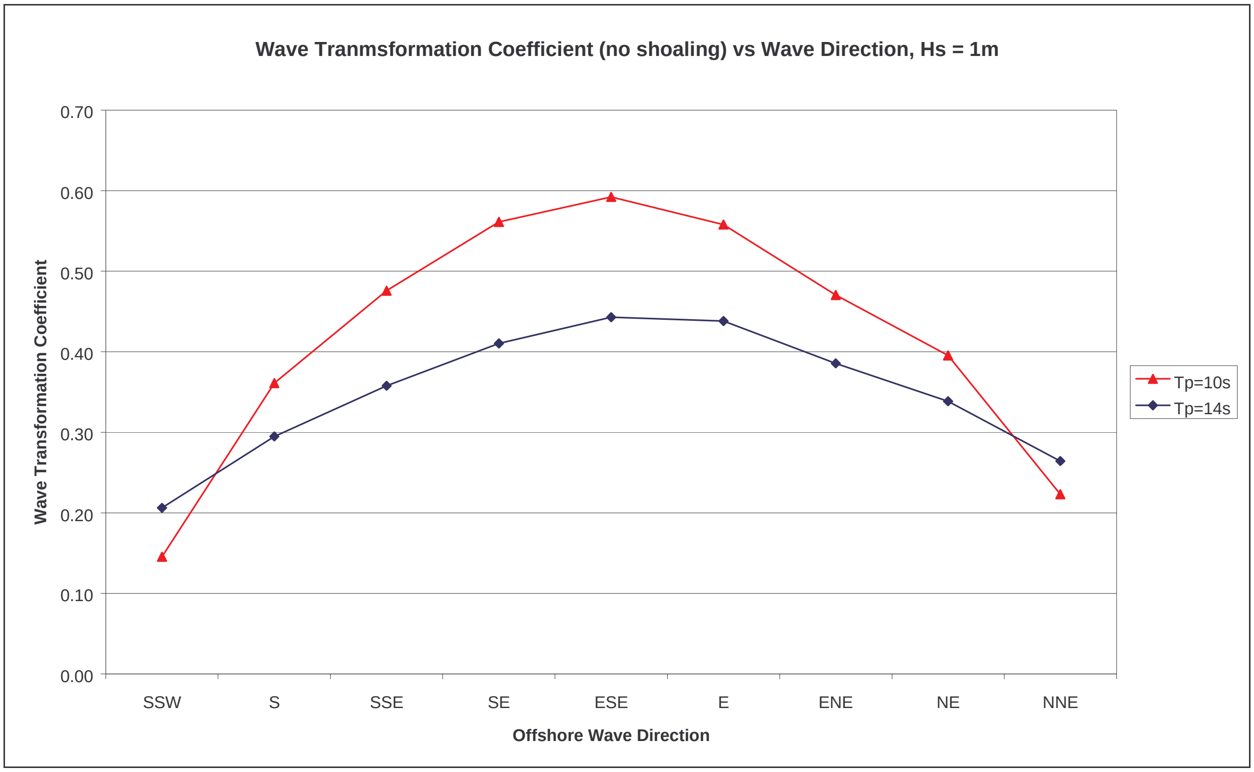


Figure 4.1 – SWAN Wave Transformation Coefficient vs Offshore Wave Direction

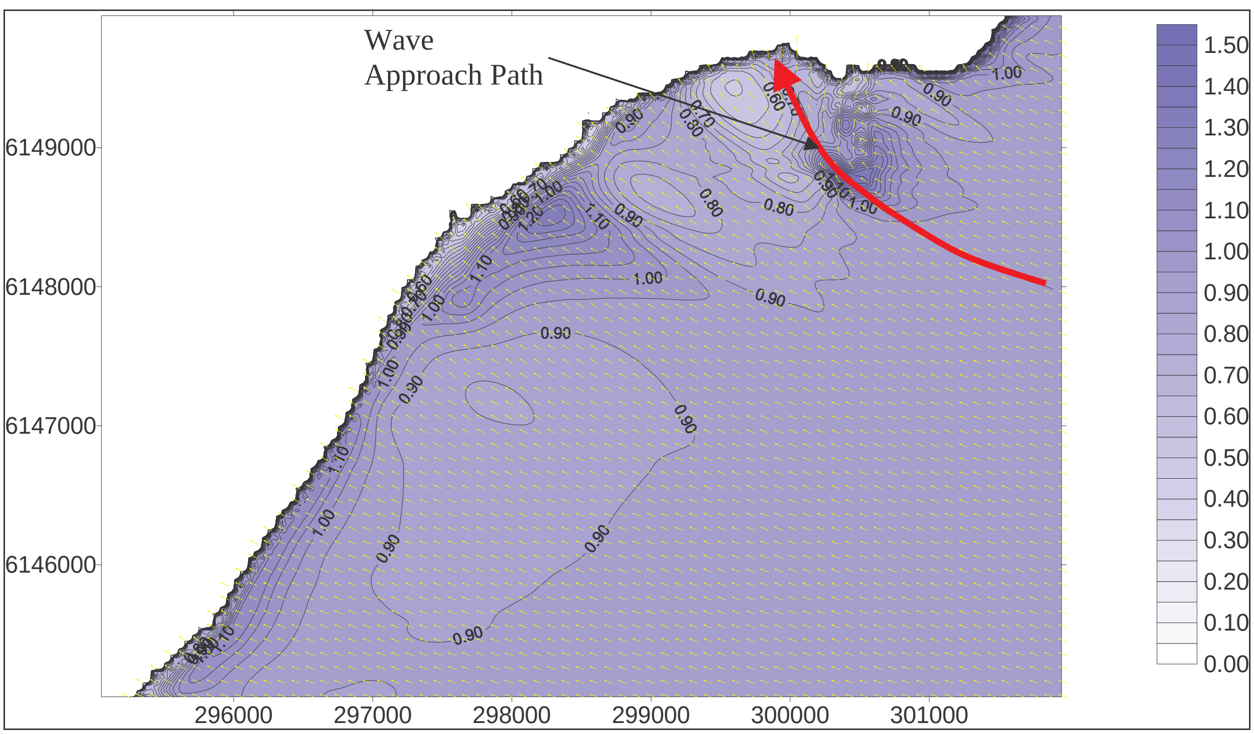


Figure 4.2 – Wave Transformation Coefficients, Vectors and Wave Approach Path for Hs=1m, T_p=10s, ESE offshore direction

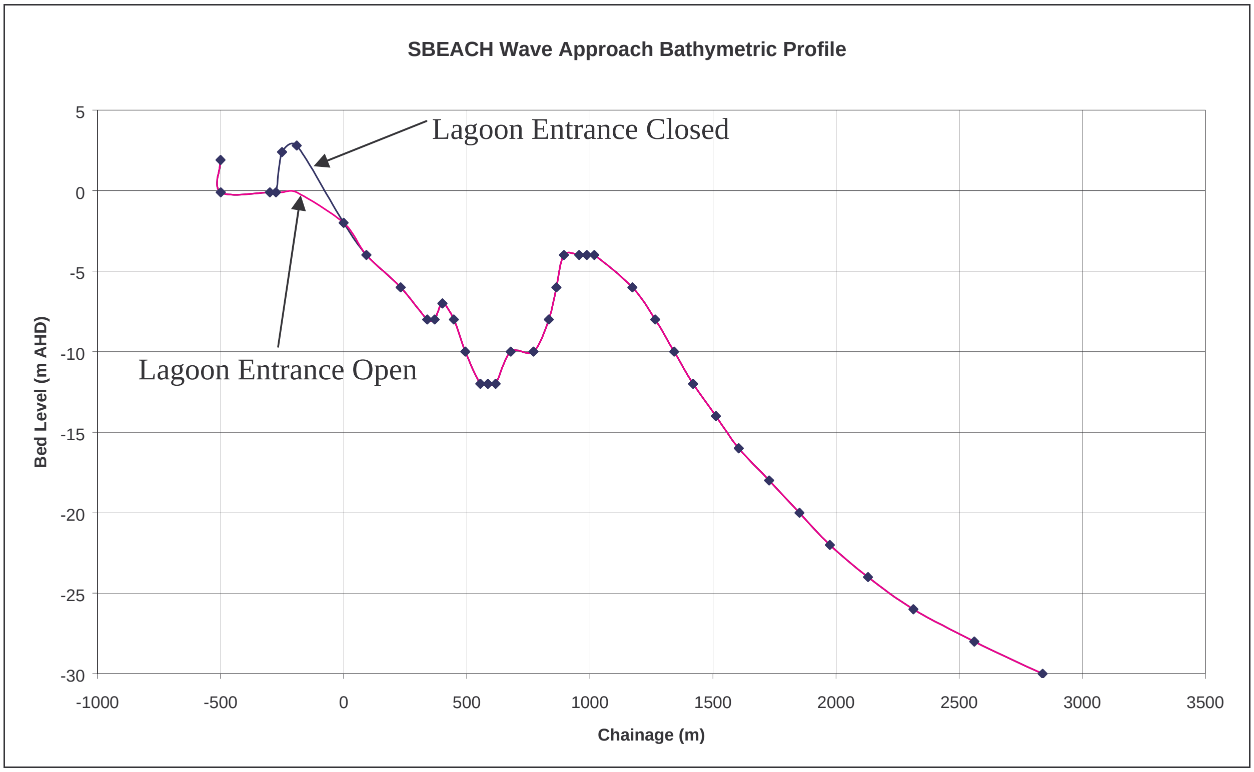


Figure 4.3 – Bed Profile along path of wave approach, used in SBEACH model

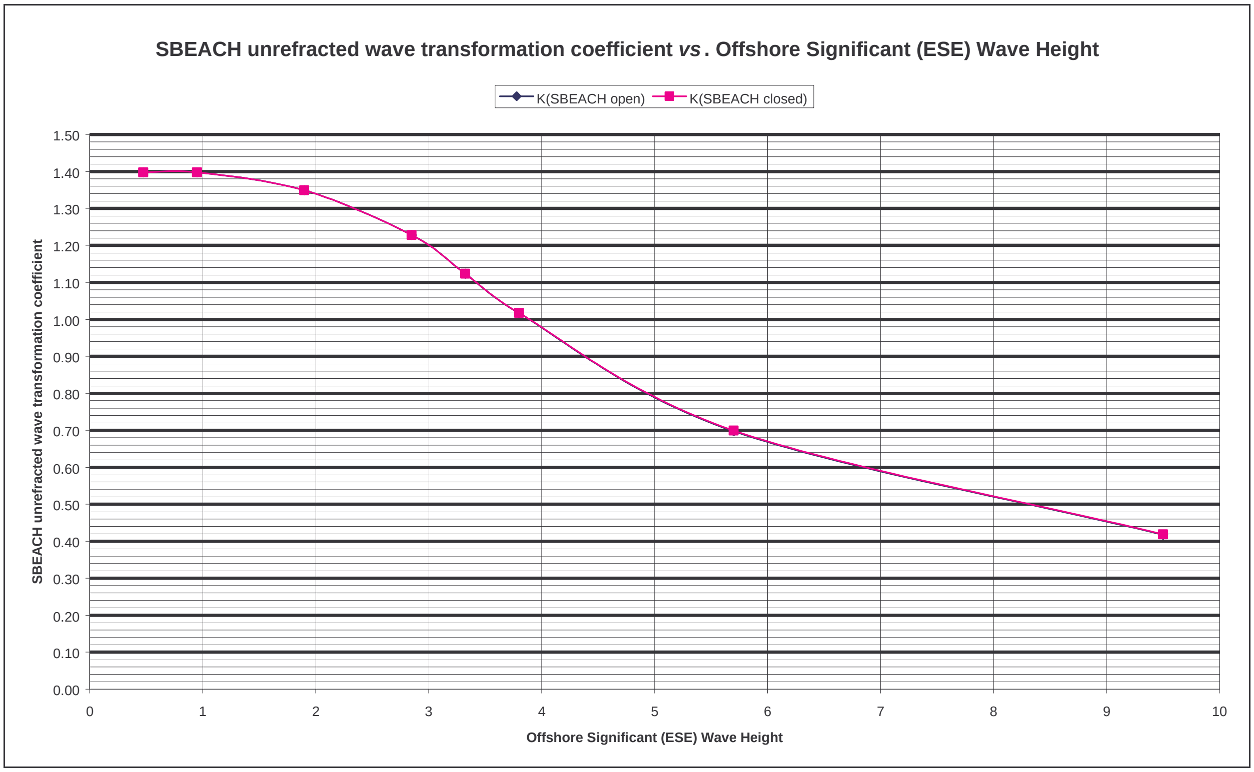


Figure 4.4 – SBEACH unrefracted wave transformation coefficient vs. Maximum unrefracted wave height

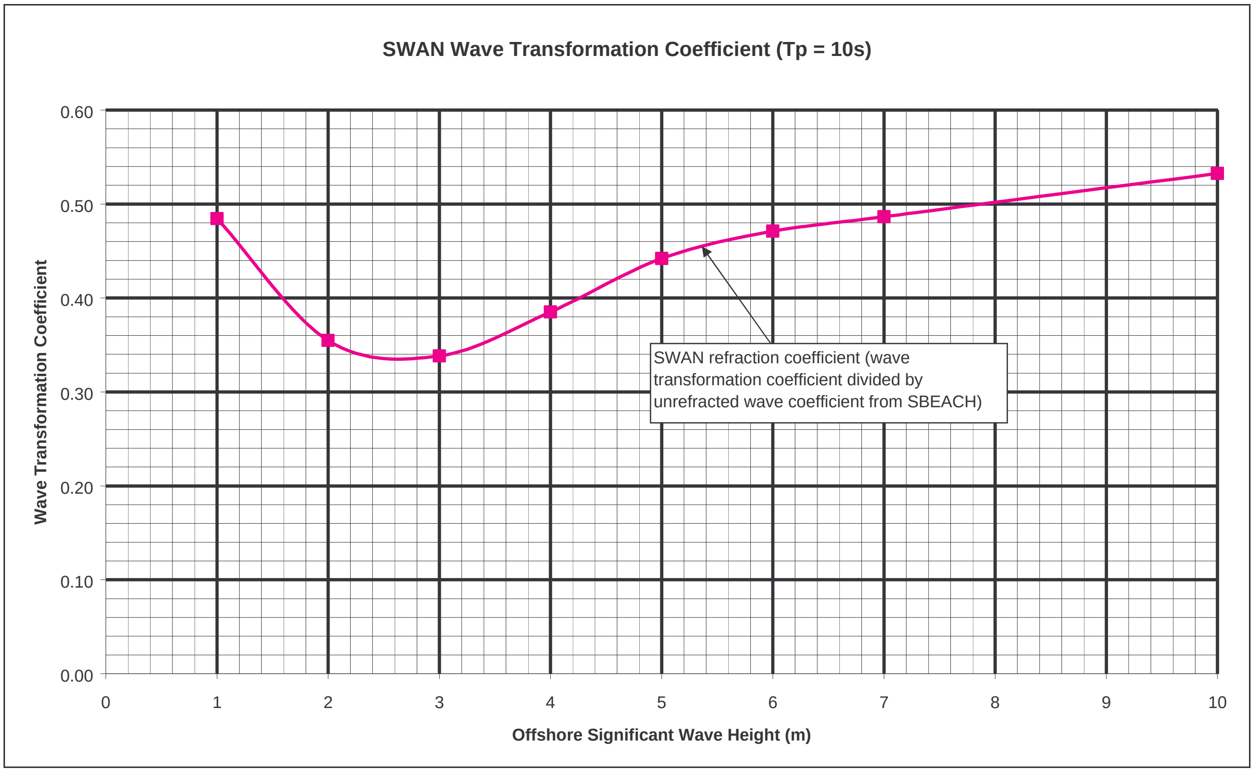


Figure 4.5 – SWAN Wave Transformation Coefficient (no shoaling) vs Offshore Significant Wave Height, for T_p=10s

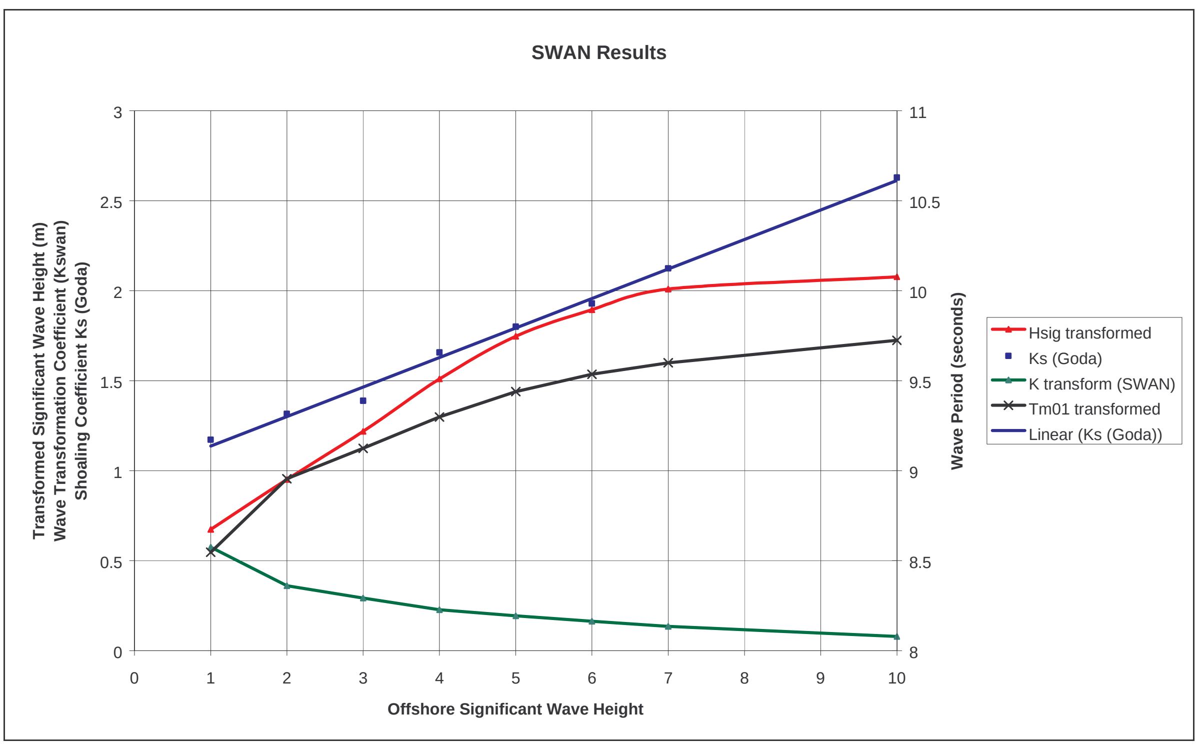


Figure 4.6 – General SWAN results as a function of offshore significant wave height, for T_p=10s and direction ESE

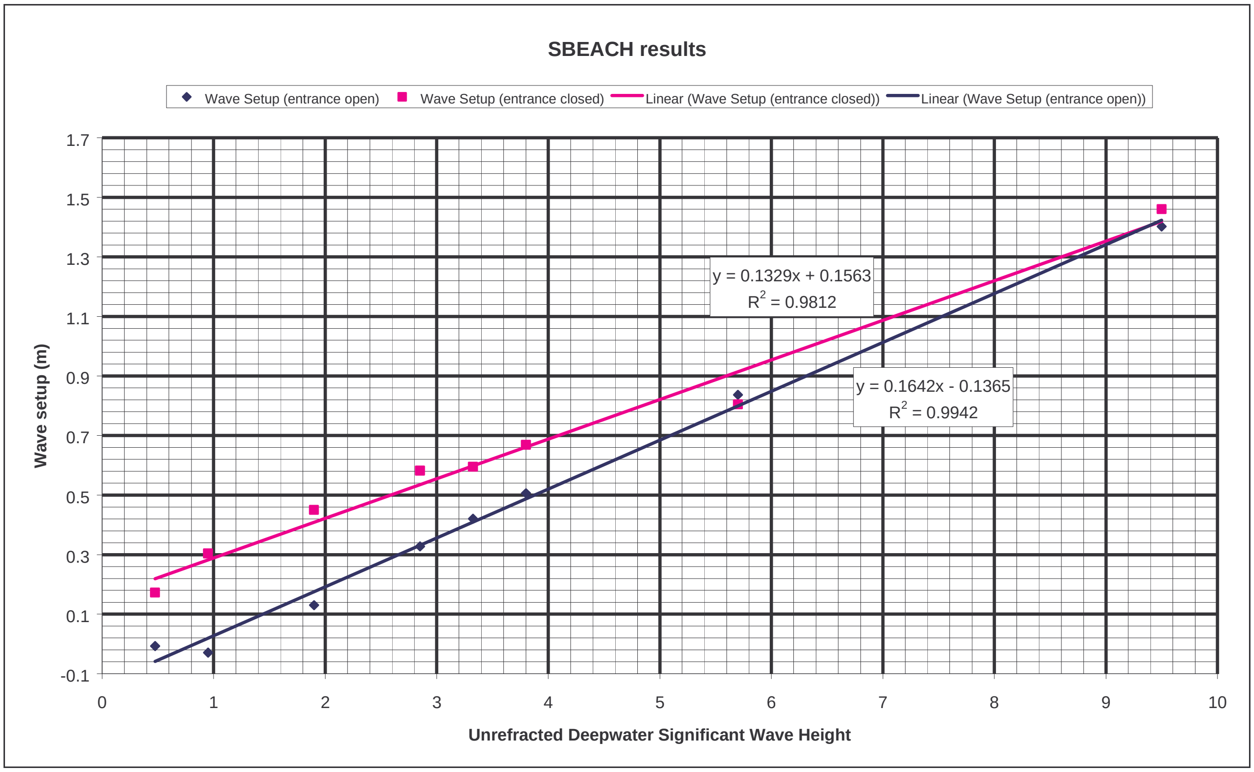


Figure 4.7 – SBEACH calculated wave setup vs Unrefracted Deepwater Significant Wave Height, Open and Closed River Entrance

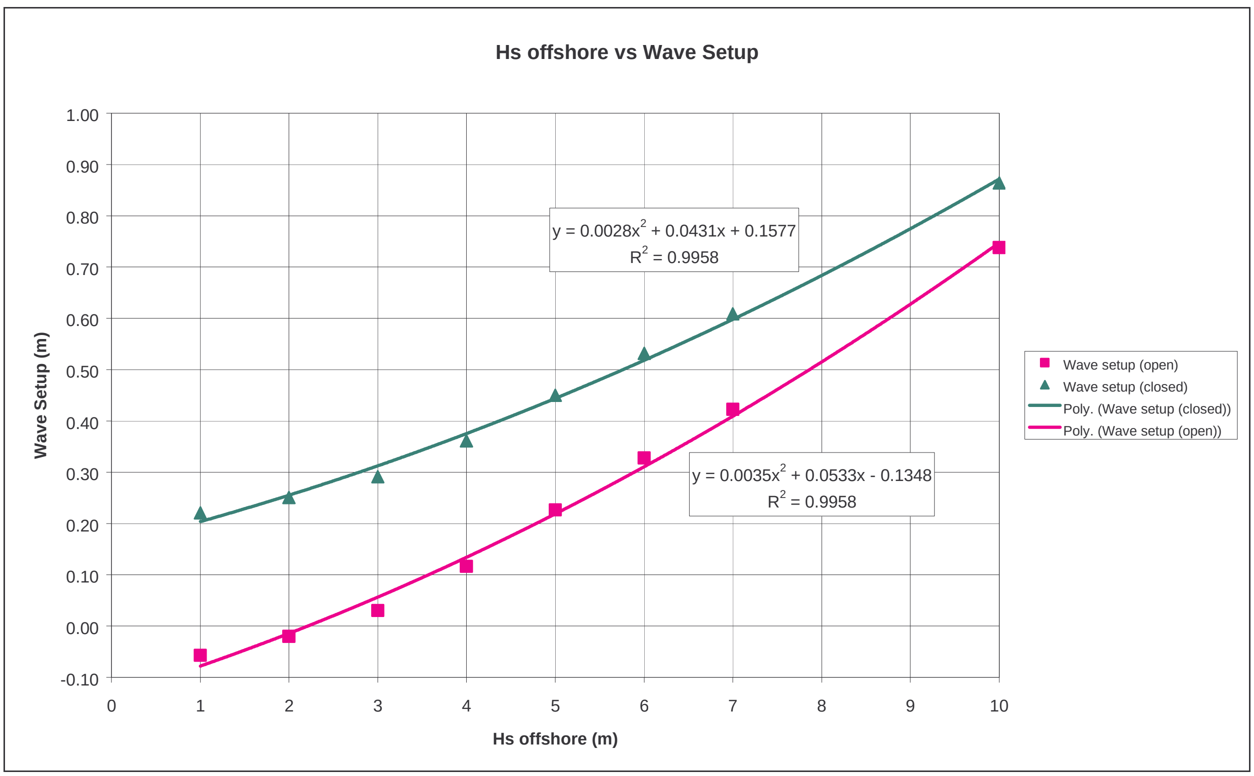


Figure 4.8 –Wave Setup vs H_s (offshore) at Crooked River Entrance for Tp=10s

Appendix B

Sediment Data

Coffey Geosciences Pty Ltd A.C.N. 056 335 516

Geotechnical | Resources | Environmental | Technical | Project Management

142 Wicks Road, North Ryde, NSW, 2113
Ph: (02) 9888 7444 Fax (02) 9878 8155

Coffey

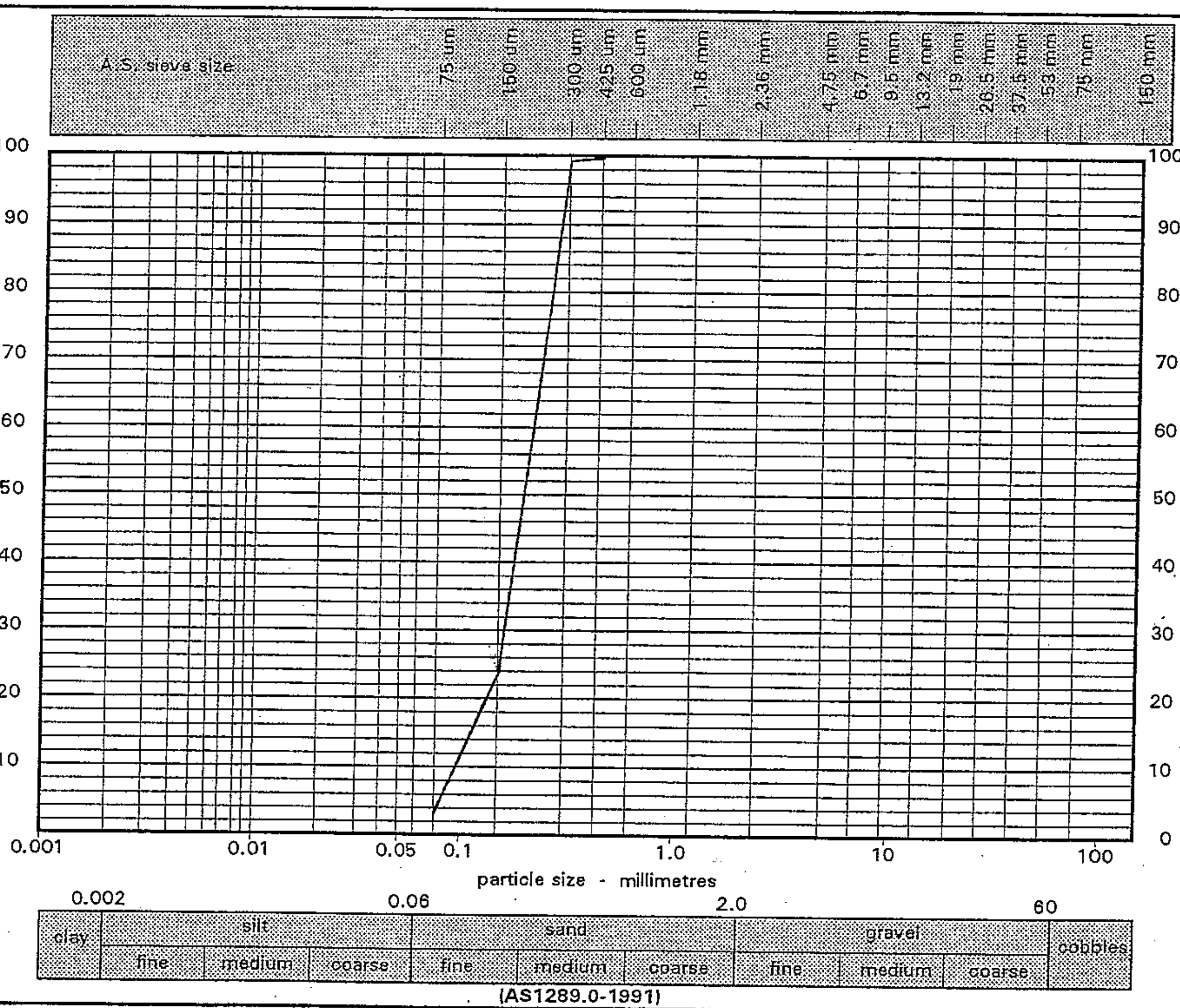
particle size distribution

client :	*SMWC AUSTRALIA*	job no :	*SL4233/1*
principal :		laboratory :	*SYDNEY*
project :	*MATERIAL ASSESSMENT*	report date :	*September 12, 2002*
location :	*CROOKED RIVER, GERROA*	test report :	

test procedure : *AS 1289.3.6.1* depth : -

sample no : *Sample No.9 (Clinet No.1)*

sample identification: *Sample supplied by the client on the 11/9/02.*



classification :

Atterberg Limit :

liquid limit	%	-
plastic limit	%	-
plasticity index	%	-
linear shrinkage	%	-
natural moisture	%	-

Sample History

- natural state ☐
- air dried ☐
- oven dried ☐
- other ☐

Preparation Method

- dry sieving ☐
- wet sieving ☐

Linear Shrinkage

Mould size mm

- crumbing ☐
- curling ☐

NATA Accredited Laboratory No. 431

Authorised Signature:

James Russell
Laboratory Manager,

Date : *12 September, 2002*

Coffey Geosciences Pty Ltd A.C.N. 056 335 516

Geotechnical | Resources | Environmental | Technical | Project Management

142 Wicks Road, North Ryde, NSW, 2113
Ph: (02) 9888 7444 Fax (02) 9878 8155

Coffey

test results

client :	*SMEC AUSTRALIA*	job no :	*SL4233/1*
principal :		laboratory :	*SYDNEY*
project :	*MATERIAL ASSESSMENT*	report date :	*September 12 ,2002*
location :	*CROOKED RIVER, GERROA*	test report :	
test procedure :	*AS 1289.3.6.1*	test date :	*11/09/2002*

Sample Identification	SAMPLE NO.10 (2)	
	PERCENTAGE RETAINED	PERCENTAGE PASSING
Sieve Size - mm		
0.425	*0.0*	*100*
0.300	*0.4*	*100*
0.150	*85.8*	*14*
0.075	*12.6*	*1*
<0.075	*1.2*	*1*

remarks : *Sample supplied by the client on the 11/09/2002.*

NATA Accredited Laboratory No. 431

Date : *September 12, 2002*

Authorised Signature:

J Russell

James Russell
Laboratory Manager

Coffey Geosciences Pty Ltd A.C.N. 056 336 516

Geotechnical | Resources | Environmental | Technical | Project Management

142 Wicks Road, North Ryde, NSW, 2113
Ph: (02) 9888 7444 Fax (02) 9878 8155

Coffey

test results

client :	*SMEC AUSTRALIA*	job no :	*SL4233/1*
principal :		laboratory :	*SYDNEY*
project :	*MATERIAL ASSESSMENT*	report date :	*September 12 ,2002*
location :	*CROOKED RIVER, GERROA*	test report :	
test procedure :	*AS 1289.3.6.1*	test date :	*11/09/2002*

Sample Identification	SAMPLE NO.9 (1)	
	PERCENTAGE RETAINED	PERCENTAGE PASSING
Sieve Size - mm		
0.425	*0.0*	*100*
0.300	*0.4*	*100*
0.150	*76.0*	*24*
0.075	*20.2*	*3*
<0.075	*3.4*	*3*

remarks : *Sample supplied by the client on the 11/09/2002.*

NATA

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Date : *September 12, 2002*

Authorised Signature:

James Russell
Laboratory Manager

APPENDIX D

HYDRAULIC MODELLING

Prepared by Paterson Consultants

CLEARY BROTHERS (BOMBO) PTY LTD

APPRAISAL OF SAND MINE EXTENSION, GERROA

REPORT

September 2005

CLEARY BROTHERS (BOMBO) PTY LTD

APPRAISAL OF SAND MINE EXTENSION, GERROA

REPORT

September 2005

Prepared By:

PATERSON CONSULTANTS PTY LIMITED
1/45 PRINCE STREET
GRAFTON NSW 2460

P O BOX 596
GRAFTON NSW 2460

Tel: (02) 6643 1588
Fax: (02) 6642 7566
E-mail: patcon@nor.com.au

Authorised for Release

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3.	CONCLUSIONS	10

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2.	Affluxes Created - Proposed Filling for Golf Course	8
3.	Design Peak Flows	9

FIGURES

1. Site Plan
2. MIKE-11 Layout

1. INTRODUCTION

Cleary Brothers (Bombo) Pty Ltd operate a sand mine just north of Berry Beach Road, Gerroa.

The mine adjoins a wide low lying area of land to its north, identified as Foys Swamp. Foys Swamp drains northward behind the coastal dune system to the Crooked River and thence to the Pacific Ocean.

Foys Swamp has been drained (by construction of a series of lateral drains) and has been used primarily for grazing.

The general location of Foys Swamp, in relation to Crooked River and Gerroa, is shown on Figure 1.

Cleary Brothers are proposing to extend their existing sand mining operation further north.

The location of the proposed extension of the sand mine is also illustrated on Figure 1. The area affected lies between Blue Angle Creek and the coastal road south from Gerroa. The mining process involves removal of overburden and construction of bund walls around the mine site. Thus, the mine site is effectively removed from the floodplain area.

Thus, a possible consequence of the extension of the mine site is to increase existing flood levels in Foys Swamp (through loss of flood storage) and along Blue Angle Creek (through loss of flood conveyance).

The object of this report is to quantify any changes to existing flood behaviour that the proposed mine extension may have.

A major flood investigation into Crooked River and Blue Angle Creek (which runs from Foys Swamp to Crooked River) has been undertaken. The investigation was reported in "Blue Angle Creek Flood Study" - Perrens Consultants (1992).

The Flood Study provided flood level estimates at the Cleary Brothers sand mine site for floods with return periods of 5, 10, 20, 50 and 100 years Average Recurrence Interval (ARI) and Probable Maximum Flood (PMF).

The Flood Study established the above flood levels by a process of:

- **establishment of a catchment flood runoff model using RAFTS;**
- **establishment of an hydraulic model of Foys Swamp, Blue Angle Creek and Crooked River using the Danish Hydraulics Institute package, MIKE-11;**
- **assessment of the break out process for Crooked River through the beach berm using the dam break and erosion routines in MIKE-11;**

- **establishment of appropriate water levels in the ocean at the beach zone as tailwater for the MIKE-11 model. This process included assessment of storm surge components, wave refraction and set up on the beach face.**

Within the above, it should be noted that:

- **Crooked River is an intermittently open and closed lagoon which appears to be "open" more than "closed";**

- **the area investigated with MIKE-11 essentially covered Foys Swamp, Crooked River and their major tributaries east of the Southern Railway Line;**

- **topographic data for the MIKE-11 model was provided by ground survey, sourced as:**

    - **hydrographic survey (as a grid) of the surf zone, beach and offshore provided by DPWS surveys;**

    - **hydrographic survey (as cross-sections) of Crooked River, provided by DPWS survey;**

    - **cross-section survey of Foys Swamp, Blue Angle Creek and other parts of the Crooked River floodplain by ground survey specifically for that project.**

It should be noted that site investigations, enquiries with Kiama Council and DLWC did not reveal a significant data base of historical flood levels.

The combination of the catchment hydrology model and the hydraulic model (MIKE-11) established in the Flood Study are appropriate techniques to examine the potential change to flood levels following the expansion of the mine site.

As noted, this report identifies the changes to flood behaviour if the sand mine is expanded as proposed.

Chapter 2 outlines the analysis undertaken, while Chapter 3 covers conclusions.

2. ANALYSIS

The Blue Angle Flood Study (Perrens, 1992) indicated:

- **Foys Swamp tended to act as a "level pool" during flooding, with floods exiting via Blue Angle Creek;**
- **rainfall over the Foys Swamp catchment was the main factor controlling flood levels within Foys Swamp, while the prevailing ocean levels and entrance conditions were of lesser importance;**
- **for comparative purposes:**
    - **the 36 hour design storm was "critical";**
    - **the tailwater (ocean water level) was set at RL 0.6 m AHD, constant with time;**
    - **the Crooked River entrance was set as "open".**

It is noted that:

- **the development proposal is generally located as shown on Figure 1;**
- **the schematic diagram of the MIKE-11 layout is shown on Figure 2;**
- **given that MIKE-11 uses waterway areas for calculation, additional cross-sections have been surveyed to ensure correct representation of the mine extensions and their impact on waterway areas.**

Three scenarios for the topography have been examined, namely:

- **Topography as existed in 2002 (thus representing the mine as existed in 2002);**
- **Current topography (and thus indicating the current mine development);**
- **Proposed topography after the proposed extension of the mine.**

The MIKE-11 model was tested for each scenario to ensure that any changes that might occur because of the insertion of additional survey are accounted.

Predicted flood levels for Foys Swamp are shown in Table 1 below for:

- **the three scenarios for topography, namely "2002", "existing 2005" and "proposed extension";**
- **"design" 36 hour storm with return periods of 100, 50, 20, 10 and 5 year ARI;**

- constant ocean level set at 0.6 m AHD;

- six references, annotated as "A" to "F" inclusive, as shown on Figure 1.

The principal interest for the present investigation is the increase in flood levels, or afflux, from existing conditions to "after mine extension" (termed "proposed" in Table 1).

Table 1

Predicted Flood Levels - Foys Swamp

Run	Topography	Hydrology	Tide	Flood Level at Location (m AHD)					
				A	B	C	D	E	F
1	Topo - 2002	100 yr - 36 hr	0.6 m Constant	2.631	2.631	2.629	2.628	2.615	1.7
2	Existing - 2005	"	"	2.644	2.645	2.643	2.641	2.624	1.7
3	Proposed	"	"	2.659	2.660	2.658	2.654	2.638	
4	Topo - 2002	50 yr - 36 hr	0.6 m Constant	2.195	2.1906	2.194	2.193	2.174	1.7
5	Existing - 2005	"	"	2.201	2.202	2.200	2.199	2.176	1.7
6	Proposed	"	"	2.207	2.208	2.206	2.200	2.181	
7	Topo - 2002	20 yr - 36 hr	0.6 m Constant	2.10		2.10	2.099	2.081	1.7
8	Existing - 2005	"	"	2.106		2.105	2.104	2.082	1.7
9	Proposed	"	"	2.111		2.111	2.103	2.086	
10	Topo - 2002	10 yr - 36 hr	0.6 m Constant	2.022		2.021	2.021	2.003	
11	Existing	"	"	2.026		2.025	2.024	2.003	1.7
12	Proposed	"	"	2.030		2.029	2.022	2.011	
13	Topo - 2002	5 yr - 36 hr	0.6 m Constant	1.96		1.96	1.959	1.942	
14	Existing - 2005	"		1.963		1.962	1.962	1.942	
15	Proposed	"		1.967		1.966	1.961	1.954	

The affluxes created by moving from the 2002 topographical situation to the current (2005) mine configuration, then transition to the proposed mine extension, are tabulated in Table 2. Examination shows that the affluxes created range between 0.028 metres to 0.007 metres, depending on the flood frequency adopted. Affluxes of this magnitude are simply not measurable or significant in a practical sense.

Table 2

Affluxes Created - Proposed Extension of Sand Mine

Original Flood Level (m AHD)	Flood Frequency	Fill Scenario	Afflux (m)
2.631	1:100 yr ARI	Existing mine	0.013
		Proposed mine	0.028
2.195	1:50 yr ARI	Existing mine	0.006
		Proposed mine	0.012
2.100	1:20 yr ARI	Existing mine	0.006
		Proposed mine	0.011
2.022	1:10 yr ARI	Existing mine	0.004
		Proposed mine	0.008
1.960	1:5 yr ARI	Existing mine	0.003
		Proposed mine	0.007

Table 3 below indicates the peak flow at the access road culvert (which is fitted with flap gates). Examination of Table 3 shows that the transition from the 2002 topography to the current (2005) mine site and from the current mine site to the proposed mine site produces very little difference in the peak flows from Foys Swamp with the current conditions and the proposed extension to the sand mine.

<u>Table 3</u>

Design Peak Flows

Run	Topography	Hydrology	Tide	Peak Flow at Culvert (cu m/sec)
1	Topo-2002	100 yr - 36 hr	0.6 m constant	26.2
2	Existing-2005	"	" "	26.3
3	Proposed	"	" "	26.5
4	Topo-2002	50 yr - 36 hr	" "	14.3
5	Existing-2005	"	" "	14.2
6	Proposed	"	" "	14.3

3. CONCLUSIONS

Conclusions that can be drawn from this investigation are:

- **MIKE-11, which is an unsteady state model, is appropriate to list the change in flood levels that would be created by the proposed extension of the sand mine;**

- **the affluxes (change in flood levels) created increase as the design floods become larger (this is an expected result);**

- **the affluxes created are small and would not be measurable.**

FIGURES

CLEARY BROTHERS (BOMBO) PTY LTD
PROPOSED SAND MINE EXTENSION

LEGEND
Survey: PWD for new STP
Survey: Cleary Brothers (2003)
Survey: Cleary Brothers (2005)

Sand mine, pond area 2003
Sand mine, working area 2005
Sand mine, proposed extension

Reference Points A

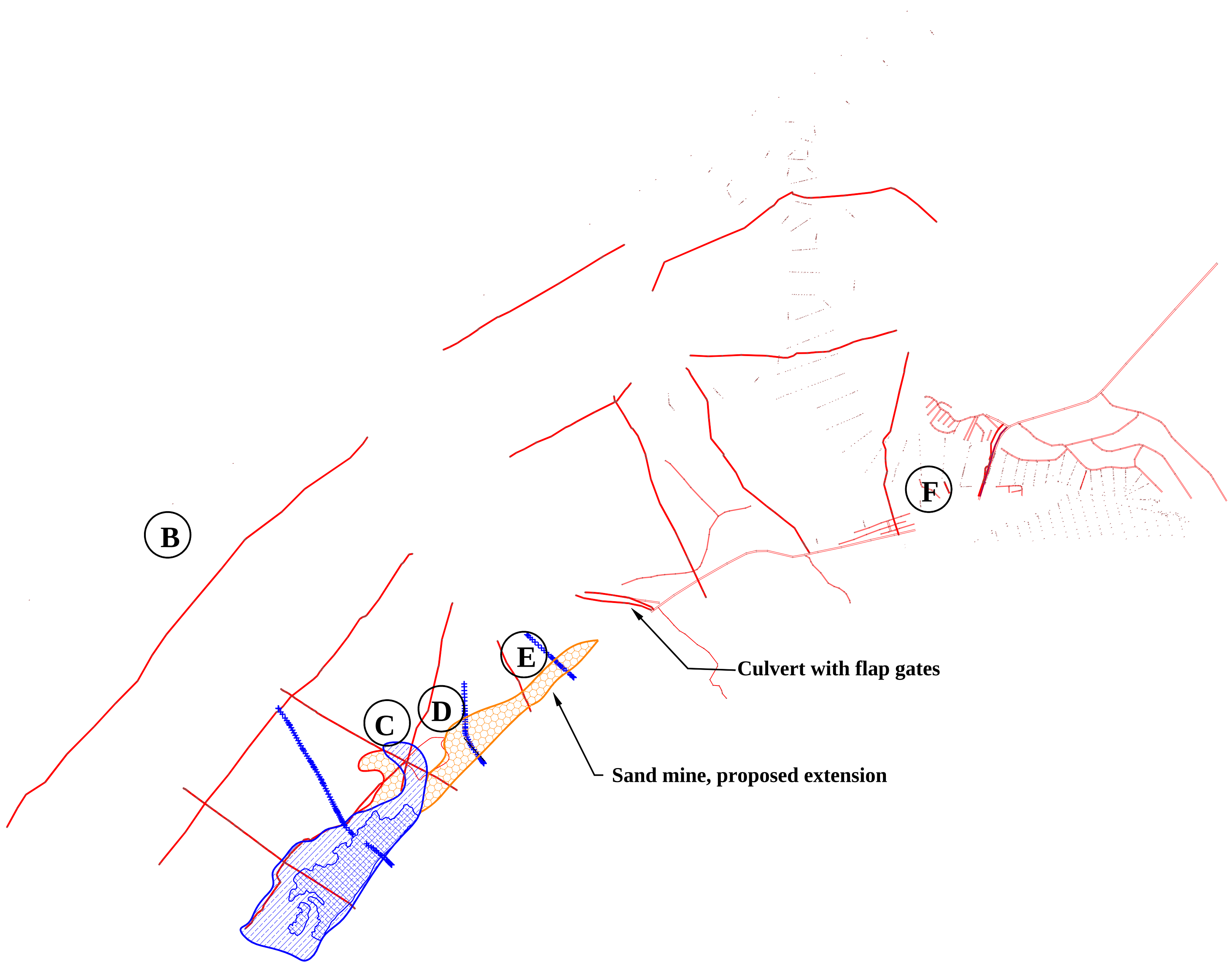
0 1000 2000
METRES

SCALE 1 : 25,000

5 SEPT 2005
DISK REF: 05029
FIG REF: 05029_FIG1_V1

FIGURE 1
SITE PLAN

CLEARY BROTHERS (BOMBO) PTY LTD
PROPOSED SAND MINE EXTENSION



LEGEND
Survey: PWD for new STP
Survey: Cleary Brothers (2003)
Survey: Cleary Brothers (2005)

Sand mine, pond area 2003
Sand mine, working area 2005
Sand mine, proposed extension

Reference Points A

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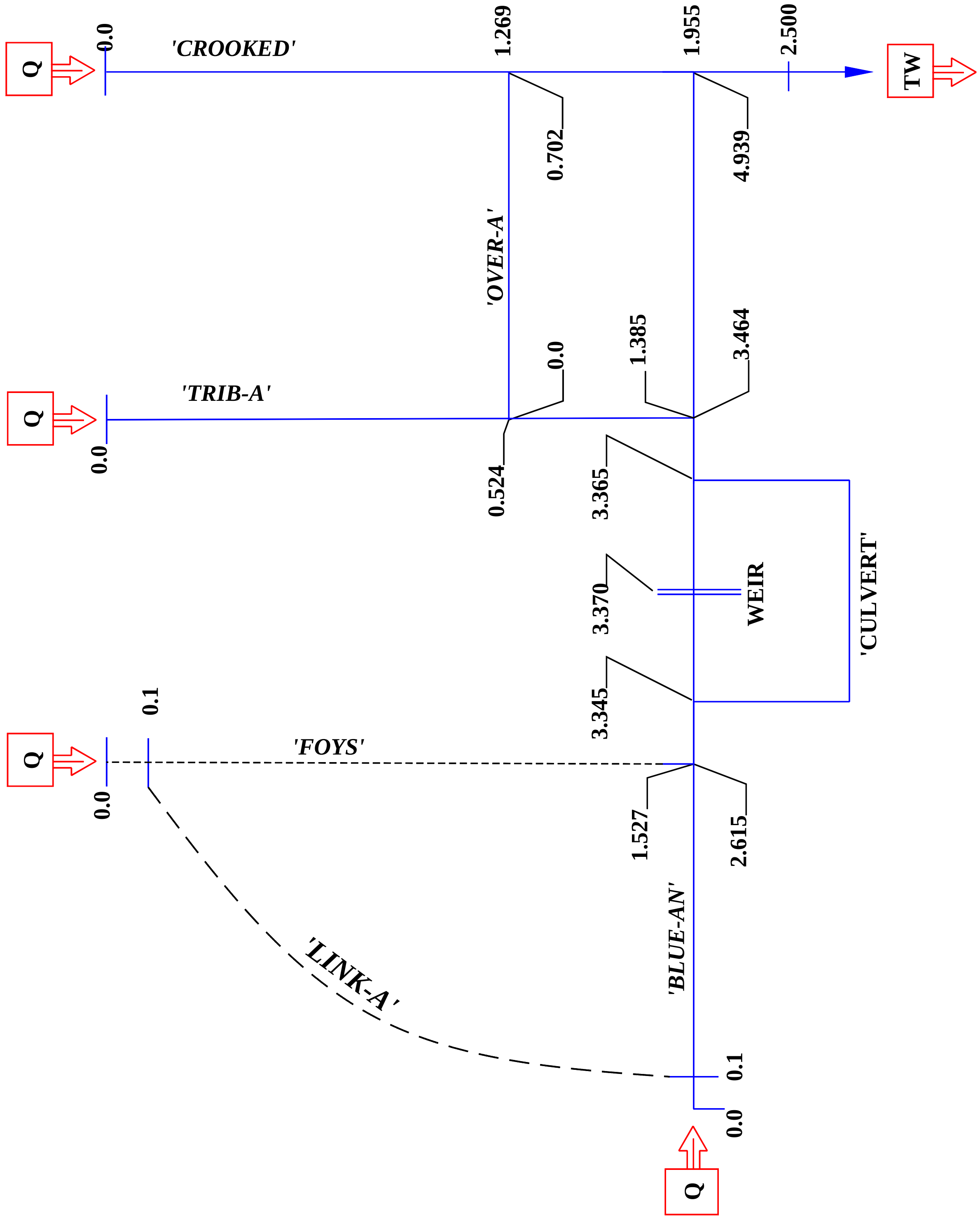
FIGURE 1
SITE PLAN

CLEARY BROTHERS (BOMBO) PTY LTD
PROPOSED SAND MINE EXTENSION



6 SEPT 2005
DISK REF: 05029_GERROA
FIG REF: 05029_Fig2_M11Schematic

FIGURE 2
MIKE-11 NETWORK