LAND & WATER CONSERVATION

QUEENSLAND GOVERNMENT
Department of Environment

Tweed River Entrance Sand Bypassing Project

Permanent Bypassing System

Technical Appendix II: Coastal Process Modelling

Hyder Consulting

Patterson Britton & Partners Pty Ltd

WBM OCEANICS AUSTRALIA

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FOR: New South Wales Department of Land and Water Conservation

and

Queensland Department of Environment

BY: WBM Oceanics Australia on behalf of:
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1 General Considerations and Methodology

The processes involved in the longshore transport of sand past the Tweed River entrance, around Point Danger and along the southern Gold Coast beaches are extremely complex. To quantify these processes for this study project, it has been necessary to develop an innovative modelling procedure combining analyses of propagation, breaking and radiation stresses of deep water wave conditions with computation of nearshore currents and sand transport processes to represent the existing sand transport regime and its spatial and temporal variability.

In this case, comprehensive two-dimensional (in plan) computer modelling has been adopted to properly represent the inter-related action of the tide, winds and wave forces in the region in generating the complex current patterns within and outside the surfzone, at the Tweed River entrance and around the headlands and structures. In addition, linking of the two-dimensional coastal zone model to the existing networked one-dimensional model of the Tweed River estuary allowed comprehensive and dynamic modelling of the whole tidal system.

Analysis of wave propagation and longshore sand transport for each of the beach sites provides for:

- comprehensive wave propagation from the range of prevailing directions for the range of incident heights and periods incorporating combined refraction, diffraction and bed friction processes.
- generation of nearshore currents generated by the combined action of tidal, wind and wave radiation stress influences.
- calculation of the spatial patterns of longshore sand transport rates resulting from the combined action of waves and currents within and outside the surfzone areas at each location.
- calculation of longshore sand transport rates in time-series form, to facilitate analysis of the temporal variability of transport at each site.

This modelling utilised the WBM Oceanics Australia software packages TWOPRO, TUFLOW and RCPR, incorporating the wave propagation program RCPWAVE as modified and adopted for the project. Software developed specifically for this project has been used to generate the time series output results.

As well, new software has been developed for assessment of cross-shore sand transport rates in time series form, and to allow results to be obtained for various locations across the nearshore profiles out to deep water. Thus, the temporal and spatial variability of shore-normal transport has also been determined.

Beach evolution modelling has been undertaken using the CERC package GENESIS. This utilises the simpler CERC equation for longshore transport, not ideally suited to this complex area, but useful as a practical indicator of beach responses to natural wave variations and the proposed bypassing works.

A description of the coastal and ocean process modelling undertaken for this project is set out in the following sections. Where appropriate, assessments are made with respect to environmental impacts, discussed further in the Environmental Impact Assessment document, and to performance criteria for the sand bypassing operations, also described further in the separate relevant study report.

2 Waves

2.1 General Considerations

Necessary input to the modelling is comprehensive wave data. The Tweed/Gold Coast region is subject to a moderate to high energy ocean wave climate with significant seasonal variability.

For the present purposes, this takes the form of a time series of height, period and direction for both 'sea' and 'swell' waves. Previous data collection and hindcasting has shown that 'sea' waves generated by local winds and 'swell' waves propagating from more distant sources commonly coexist in the study region. They have been provided for independently and in combination because:

- sea and swell often propagate from different directions and have different effects on longshore sand transport.
- the influence on currents of stronger winds associated with higher sea waves is significant in some areas.

While an extensive data record of non-direction wave information is available for the region, fully directionally recorded wave data is available for only limited periods at locations relatively nearshore (in 20 to 25 metres water depth) at Gold Coast Seaway and Letitia Spit. A considerably longer directional wave data base is needed to understand the longer term spatial and temporal variability of the sand transport processes. It was therefore necessary to establish directional information to augment the recorded wave data for use in the study.

2.2 Non-Directional Wave Climate

2.2.1 Recorded Data

Non-directional wave data for the region has been sourced from the recorded and observed data. The primary source has been the Queensland Beach Protection Authority's (BPA's) wave data acquisition program. This includes non-directional data recorded at Brisbane, Gold Coast, and Kirra and directional data recorded at the Gold Coast. In addition, wave data from Byron Bay has been made available by the Department of Land and Water Conservation (DLWC) in NSW.

The locations of all these sites are shown in Figure 2.1.

BPA reports describing and analysing the recorded non-directional wave climate at Brisbane (1976 - 1994), Gold Coast (1987 - 1994) and Kirra (1988 - 1994) have been published as data reports W09.2, W14.1 and W15.1 respectively. The recording and analysis techniques described in these reports continues to date and the data set 01/01/89 to 30/06/96 has been chosen for this project.

Similarly a report describing the recorded and analysed wave data for Byron Bay is published annually by DLAWC and the data set 01/01/89 to 30/06/96 was used in calibrating the final composite time series.

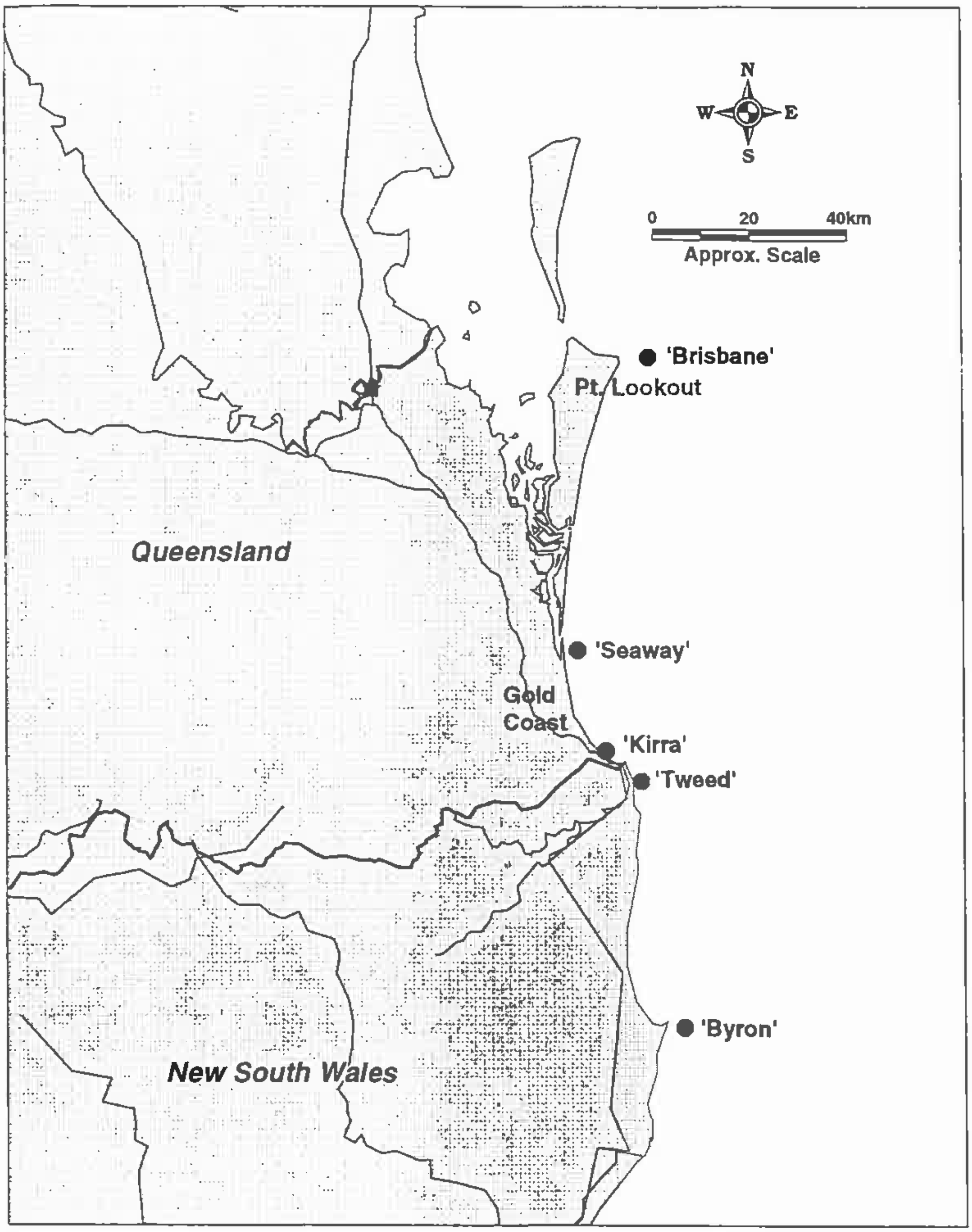


Figure 2.1
Locations of Wave Recording Sites

Source :

2.3 Directional Wave Climate Assessment

2.3.1 General Approach

Directional wave data is essential in modelling and assessment of sediment transport in the nearshore coastal zone. No reliably recorded medium to long term directional wave climate data is available for use in this study. Discussion of the status of determination of a directional wave climate for the region is best summarised in the recent report by Delft Hydraulics (Roelvink and Murray 1992). That report highlights the deficiency of the available data and the implications for the computation of longshore transport.

It has thus been necessary to utilise a combination of the recorded non-directional data and hindcast directional information to obtain a suitable basis for the process investigations. As part of this study, considerable effort has been put into determining appropriate deep water wave directions for use with the recorded wave height and period data as input to sediment transport modelling. The non-directional Brisbane wave data was selected as the base time series because of the length of record, depth of water at the Waverider site and proximity to the study site. It is also close to a British Meteorological Office (BMO) hindcasting model grid point.

The following sections detail the background to the selection of the final composite time series of directional wave data.

2.3.2 Available Wave Direction Data

As part of the Tweed River Entrance Sand Bypassing Project, a directional wave recording station was established in January 1995 offshore from Letitia Spit in about 20 metres water depth, and its operation is continuing. The only other source of recorded directional wave data for the region is a relatively short period (12 months from September 1991 to September 1992) obtained at a site offshore from the Gold Coast Seaway. To date no reports have been published on the directional Waverider deployments at either site. Because of its short duration, this data is of limited use for the present investigation other than for validation of the hindcasting model.

Apart from these relatively short periods of recorded directional data (2.5 years in approximately 50 years of recorded data), all of the existing deep water directional wave data is a composite of recorded wave height and period and assessed wave directions. To date, the assessed wave directions have been derived by relatively simple correlation with synoptic charts. This is not as comprehensive or reliable as regional wave hindcasting taking into account both sea and swell. Hindcasting of this nature is carried out by the British Meteorological Office (BMO), although at a relatively coarse resolution in this region.

The BMO hindcasting uses an atmospheric model to calculate wind fields around the world as an aid to its climatic predictions in the North Sea and the Atlantic Ocean. As part of this modelling package, wind wave generation, propagation and decay are modelled also on a worldwide basis. For the Pacific Ocean this model uses a 1.25 degree latitude by 1.25 degree longitude grid and a water depth of 200m. The model has not been calibrated against observed conditions in this region. Sea, swell and wind parameters are available at 6 hourly time increments.

Directional information in the form of observations (COPE) and hindcasting analyses are also available for the southern Gold Coast and Brisbane respectively. While both of these methods provide reasonable statistical information on general conditions they are not accurate enough for input into sediment transport models where direction is a primary factor for daily variability of conditions.

2.3.3 Assessment of Available Wave Direction Data

Direct Recording

It is preferable to use recorded directional wave data from calibrated instruments close to the site as the primary input for the sediment transport models. However, the only directly recorded directional wave data for the Tweed region is from relatively sheltered and shallow water and is of too limited duration to be representative of the longer situation. It can be used however for input to short term local process modelling, and in the longer term will provide an invaluable data source for management of the sand bypassing system.

In the present situation, the best available option is to combine longer term recorded height and period data from Brisbane with an assessed or modelled directional component. The available options for the directional component are listed above and a brief description of their suitability is given below.

Simple Synoptic Hindcasting

Hindcast techniques using a combination of synoptic charts, local knowledge and recorded wave heights and periods have been employed to determine the wave directions most appropriate for the Brisbane wave recording site. The restrictions in this process are the regional nature of synoptic charts, a lack of information to determine long swell from distant sources (particularly Southern Ocean and Pacific swell which is generated outside the region of the charts) and the extent of manual input into the process. This data exists for the Brisbane site but preliminary comparisons with recorded wave directions carried out by the BPA showed results that were too inconsistent for daily sediment transport modelling.

COPE

The COPE (Coastal Observation Program Engineering) is operated by the BPA and uses volunteer observers to make daily observations of wind, waves and longshore current at each site. Several sites have been established on the southern Gold Coast and have varying lengths of records. The wave directions observed by the COPE volunteers are restricted because the observers are at beach level and therefore can only see wave directions in the nearshore zone after a significant amount of refraction and diffraction. Also, the observers are looking directly at the wave crest as it approaches, making the estimation of direction a difficult parallax exercise. The resulting observed wave directions are typically distributed much closer to the shore normal direction than deepwater waves.

BMO Hindcasting Model

The British Meteorological Office has been operating wind and wave models since 1976 with many revisions in the physics and algorithms until 1987 when the current generation of model was initiated. Two models are maintained, one global with the following attributes:

- 1.25 degree latitude and 1.25 degree longitude grid;
- 2 hr wind timestep;
- 6 hr 1-D wave spectrum and direction output; and
- one for Europe with higher resolution.

The current wave model uses the modelled wind field as input into wave growth, propagation and decay algorithms based on 1-D wave spectra evolution. The output carries sea and swell wave height, period and direction parameters and a resultant which is a numerical combination of the sea and swell.

Despite its limitations, this hindcasting represents the most comprehensive approach to deriving directions for joint occurrence of both sea and swell in the region, and has been used herein, with modifications aimed to correct for the inherent limitations. Discussion of this model data and its application to this project is outlined below.

2.3.4 British Meteorological Office Hindcasting

Limitations Of BMO Model

Grid Size: The global model has a grid size of 1.25 degree latitude by 1.25 degree longitude and the gridpoint chosen is 27.75 S and 154.69 E which is approximately 120km east and 55km south of the Brisbane wave recording site.

Location of Boundary: Because of the size of the grid the land boundary in the Brisbane region is significantly inland from the natural boundary and deepwater exists to that boundary. Also, no allowance has been made for the water depth limitation across the Great Barrier Reef or Continental Shelf. This means that the model is likely to over-estimate sea conditions from the west and swell propagation from the north and south.

Water Depth: The global model is not depth dependent and therefore a depth of 200m exists at all water cells. This means that no refraction or bottom friction effects are included in the model.

Modifications To BMO Model Output

Because of the limitations listed above, the output from the BMO model was modified for use in this project. These modifications relate to allowances to correct for the local geography such as the coastline shape, water depths along local fetch lengths, the Great Barrier Reef, the relative proximity to land, the depth of the wave recording sites and the BMO choice of parameters for splitting sea and swell. Firstly, some modifications were carried out to allow a reasonable comparison of the recorded (non-directional and directional) and hindcast datasets as they would be after propagation involving predominantly refraction to each of the recorder sites. Then, using this as a calibrating process, a separate set of modifications were implemented to derive a representative 'deepwater' climate based on the Brisbane recorder off Point Lookout. The modifications for the comparison and calibration of sea and swell are given below and details of the modifications for the final timeseries are given in Section 2.2.5.

Wave Periods

The BMO hindcast wave periods are presented in terms of zero crossing periods (T_Z) derived from the directional frequency spectra modelled (Ross, 1988). These results may be converted approximately to equivalent spectral peak periods (T_P) for the comparison with the recorded data based on the average relationship values derived from the recorded data (Beach Protection Authority, 1994). For sea waves, a minimum wave period equivalent to a maximum wave steepness of 0.04 was adopted for both the BMO results and where additional hindcasting was incorporated.

Sea Heights and Directions

The schematisation of the BMO hindcasting model in this region is rather coarse, with water cells (each about 90 km) of 200m deep water between land and the selected output gridpoint. The wave recording sites are actually between 2km (Kirra) and 10km (Brisbane) offshore. This excessive fetch length gives an over-estimation of the sea originating from the west in the BMO model. For this reason a separate calculation of the sea condition - using the BMO wind data for the recorder site and the depth dependent SMB method as described in the Shore Protection Manual - was included in comparisons and when calibrating the BMO output.

The physical geography of the local coastline and nearshore bathymetric shape in the study area means that processes such as fetch limitation, bottom friction, refraction and diffraction are significant. Allowances for these were incorporated by applying simplified intermediate water depth approximations for modifying the wave heights and directions.

As well, winds in the nearshore area are subject to the influences of land breeze and sea breeze effects. These effects are most significant in autumn and summer months and tend to induce offshore winds at night and onshore winds during the day. Most commonly, this manifests as a shift in northerly winds towards the northeast during the day. As well, southeast to east winds tend to move towards the northeast as they strengthen during the day. These effects produce an increased occurrence of northeast local sea waves, represented empirically in the wave direction modification routine.

Swell Height and Directions

The BMO dataset showed significant occurrences of swell propagating from the north and south directions as well as from the open ocean with unimpeded access in the sector from south-southeast to north-northeast. This is understandable, given the unrealistic land boundary location in the model schematisation. In reality, such swells either would not exist or are substantially attenuated because of the coastline shape and/or the presence of shallow water over the continental shelf, the Barrier Reef and the diffractive effects of Cape Moreton, Pt Lookout, Pt Danger and Cape Byron. Modifications relating to these effects were incorporated by decreasing those swell wave heights to account for deep water diffraction losses and modifying the directions to the most probable nearest deepwater access. Additional refraction and diffraction modifications were made when comparing with shallower water wave recording sites for data validation.

Sea/Swell Split

The BMO modelled wave is output as sea or swell depending on the wind direction. That is, if the wind was in the same direction as the wave then initially the wave was labelled as sea. For use as input into the refraction and diffraction wave model and the subsequent calculation of sediment transport, it is more appropriate to classify sea and swell in terms of steepness (Ho/Lo). A cutoff value of 0.02 was used for steepness classification to differentiate sea and swell.

By using this criterion a wave of steepness greater than 0.02 can be considered an undeveloped sea and therefore be modelled with wind. Waves of steepness less than 0.02 can be considered swell and modelled without wind.

Review Of BMO Modifications

Time Series Plots: Time series plots of the comparison of the modified BMO data with the non-directional datasets of Brisbane, Gold Coast and Kirra as well as the directional datasets of the Gold Coast (Seaway) and Tweed indicate close agreement in time series format for wave height. Even extreme events appear to be well represented by the hindcasting, despite the relatively coarse model grid. Wave period is not so well modelled but is generally of the correct order. The hindcasting typically shows occurrences of low, very long period swell not identified in the recorded data. Such swell is of little consequence for this study.

The wave direction results obtained for both the Seaway and Tweed locations show generally good qualitative agreement. However, the results are for the peak wave period and total wave height to match with the analysis of the recorded data. This has the effect of emphasising the swell component which tends to be limited in the range of incident directions in the water depths where these recorders were located. Thus, this validation check is of limited application. Further directional wave data recording in deep water and attention to the analysis of the recorded data in terms of the separate sea and swell components is recommended to more reliably assess the hindcasting.

Wave Occurrence Tabulations: Tabulations of the occurrence of height/period combinations for the recorded and hindcast information are given in Tables 2.1 and 2.2 respectively. These results again indicate reasonable correlation with the recorded wave height data, although the hindcast swell periods appear too high, leaving a deficit of occurrences in the mid-range swells.

Roelvink and Murray (1992) also utilised the BMO model to assist in enhancing the directional wave database. They concluded that the model produced results in which the swell wave occurrences are biased away from the east sector and towards the south and north sectors.

That pattern is considered to again influence the outcomes of the present studies, despite all efforts to minimise the effects of the geographical limitations inherent in the model. In that regard, none of the modifications made to the BMO results have any effect on the proportion of swell from the eastern sector which is most probably underestimated. A key feature of this deficiency is a likely excessive estimate of occurrence of southerly swell and associated upcoast sand transport along the more exposed ocean beaches, and insufficient occurrence of easterly swell and underestimated sand transport along the north-facing beaches around Kirra to Bilinga.

Considerable further directional wave recording with comprehensive analysis of the data in terms of both sea and swell components is needed to improve the presently assessed directional wave climate.

2.3.5 Adopted Deep Water Sea and Swell Time Series

The calibrations and modifications described above were to allow comparisons with recording sites which are in varying depths of water from 16m (Kirra) to 80m (Brisbane). The required wave timeseries is for deepwater conditions.

The recorded wave heights and periods for Brisbane were combined with the BMO modelled sea and swell directions, modified as detailed below, as the wave time adopted for the analysis. The final deepwater dataset is stored in computer file format (SPL_SEA2_bri38996). Ten (10) days of gaps distributed over the 7□ year period has been infilled with data taken from other concurrent datasets and fine-tuned for the Brisbane site.

Table 2.1 Recorded Non-Directional Wave Climate - Brisbane

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Spectral Peak Period (sec)								
	0-2.99	3-4.99	5-6.99	7-8.99	9-10.99	11-12.99	13-14.99	>14.99	TOTAL
0.00-0.50	0.05	0.00	0.05	0.09	0.06	0.02	0.00	0.00	0.27
0.51-1.00	0.01	1.49	2.93	7.32	7.63	2.09	0.18	0.02	21.67
1.01-1.50		1.24	5.97	14.77	12.85	2.75	0.24	0.01	37.83
1.51-2.00		0.04	3.34	8.88	7.57	1.99	0.17		22.00
2.01-2.50			0.80	5.18	3.60	1.07	0.09		10.73
2.51-3.00			0.05	2.15	1.78	0.61	0.07		4.66
3.01-3.50				0.51	0.78	0.32	0.02		1.63
3.51-4.00				0.16	0.43	0.11	0.00		0.70
4.01-4.50				0.02	0.23	0.06	0.01		0.31
4.51-5.00					0.08	0.01			0.10
5.01-5.50					0.03	0.03			0.06
5.51-6.00						0.01			0.01
6.01-6.50						0.01			0.01
6.51-7.00						0.01			0.01
7.01-7.50									0.00
TOTAL	**0.05**	**2.78**	**13.13**	**39.07**	**35.06**	**9.07**	**0.80**	**0.02**	**99.98**

Table 2.2 BMO Hindcast Non-Directional Wave Climate

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Approximate Equivalent Spectral Peak Period (sec)								
	0-2.99	3-4.99	5-6.99	7-8.99	9-10.99	11-12.99	13-14.99	>14.99	TOTAL
0.00-0.50		0.00	0.00	0.03	0.02	0.00	0.00	0.00	0.06
0.51-1.00		0.65	0.03	0.97	9.41	1.98	0.01	0.00	13.06
1.01-1.50		2.60	3.54	0.78	15.49	14.43	0.73	0.03	37.61
1.51-2.00		0.19	9.43	0.02	2.87	11.46	2.41	0.17	26.55
2.01-2.50		0.05	7.32	0.30	0.15	2.43	1.95	0.08	12.27
2.51-3.00			2.10	2.57	0.02	0.57	0.76	0.15	6.17
3.01-3.50			0.37	1.50	0.01	0.09	0.29	0.11	2.39
3.51-4.00				0.94	0.00	0.01	0.07	0.02	1.04
4.01-4.50				0.32	0.00	0.03	0.01	0.01	0.38
4.51-5.00				0.15	0.00		0.02	0.01	0.18
5.01-5.50				0.07	0.00				0.07
5.51-6.00				0.02	0.05				0.07

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Approximate Equivalent Spectral Peak Period (sec)								
	0-2.99	3-4.99	5-6.99	7-8.99	9-10.99	11-12.99	13-14.99	>14.99	TOTAL
6.01-6.50				0.02	0.05				0.07
6.51-7.00					0.05				0.05
7.01-7.50					0.02				0.02
TOTAL	0.00	3.50	22.79	7.71	28.13	31.02	6.25	0.60	99.99

Table 2.3(a) Adopted Non-Directional Swell Wave Climate

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown (Calm = 22.54%) Spectral Peak Period (sec)								
	0-2.99	3-4.99	5-6.99	7-8.99	9-10.99	11-12.99	13-14.99	>14.99	TOTAL
0.00-0.50		0.01	0.24	0.60	0.96	0.38	0.14	0.03	2.36
0.51-1.00		0.05	2.41	7.50	8.20	2.65	1.06	0.25	22.12
1.01-1.50		0.01	1.08	10.38	10.16	3.79	1.06	0.13	26.62
1.51-2.00			0.06	4.29	6.65	2.46	0.80	0.07	14.37
2.01-2.50			0.01	1.00	3.61	1.75	0.45	0.02	6.83
2.51-3.00			0.00	0.08	1.65	1.03	0.21	0.01	2.98
3.01-3.50			0.01		0.52	0.73	0.11	0.01	1.39
3.51-4.00					0.15	0.38	0.06		0.58
4.01-4.50					0.01	0.09	0.05		0.15
4.51-5.00						0.06			0.06
5.01-5.50									
5.51-6.00									
6.01-6.50									
6.51-7.00									
7.01-7.50									
TOTAL	0.00	0.07	3.81	23.86	31.91	13.30	3.93	0.52	77.46

Table 2.3(b) Adopted Non-Directional Sea Wave Climate

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown (Calm = 10.91%) Spectral Peak Period (sec)								
	0-2.99	3-4.99	5-6.99	7-8.99	9-10.99	11-12.99	13-14.99	>14.99	TOTAL
0.00-0.50	21.95	7.06	0.02	0.00	0.00	0.00			29.04
0.51-1.00	0.25	32.20	1.73	0.03	0.00	0.00			34.22
1.01-1.50		1.71	4.92	0.02	0.00	0.00			6.65
1.51-2.00		0.03	4.22	1.95	0.00	0.00			6.21
2.01-2.50			1.10	4.74	0.00	0.00			5.83
2.51-3.00			0.10	3.10	0.55	0.00			3.75
3.01-3.50			0.01	0.99	0.78	0.00			1.78
3.51-4.00				0.21	0.66	0.00			0.87
4.01-4.50				0.09	0.33	0.07			0.49
4.51-5.00					0.13	0.01			0.14
5.01-5.50					0.01	0.03			0.05
5.51-6.00					0.01	0.01			0.02
6.01-6.50						0.00			0.00
6.51-7.00						0.02			0.02
7.01-7.50						0.02			0.02
TOTAL	22.20	41.01	12.11	11.14	2.46	0.17	0.00	0.00	89.09

Table 2.3(c) Non-Directional Wave Climate - Sea plus Swell using Peak Wave Train

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Spectral Peak Period (sec)								
	0-2.99	3-4.99	5-6.99	7-8.99	9-10.99	11-12.99	13-14.99	>14.99	TOTAL
0.00-0.50	0.01		0.01	0.01	0.00	0.00	0.00	0.00	0.03
0.51-1.00	0.01	1.37	1.76	2.97	2.94	0.95	0.39	0.08	10.46
1.01-1.50		2.59	5.94	11.65	10.60	3.51	1.07	0.21	35.58
1.51-2.00		0.24	4.59	8.36	8.50	3.08	0.99	0.06	25.87
2.01-2.50			1.22	6.20	4.75	2.16	0.64	0.05	15.00
2.51-3.00			0.14	3.24	2.49	1.20	0.25	0.01	7.33
3.01-3.50			0.02	0.99	1.39	0.76	0.13	0.02	3.32
3.51-4.00				0.21	0.80	0.37	0.07		1.44
4.01-4.50				0.09	0.35	0.17	0.05		0.66
4.51-5.00					0.13	0.07			0.19
5.01-5.50					0.01	0.03			0.05
5.51-6.00					0.01	0.01			0.02
6.01-6.50						0.00			0.00
6.51-7.00						0.02			0.02
7.01-7.50						0.02			0.02
TOTAL	**0.02**	**4.20**	**13.68**	**33.72**	**31.97**	**12.36**	**3.58**	**0.42**	**99.99**

Table 2.4(a) Adopted Directional Swell Wave Climate

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Spectral Peak Period (sec)									
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	TOTAL
0.00-0.50	4.45	2.38	1.29	1.38	2.33	1.64	2.16	3.31	4.95	27.02
0.51-1.00	2.16	1.77	1.07	1.43	2.68	1.88	1.95	2.78	3.46	21.60
1.01-1.50	1.43	1.30	0.96	2.10	3.50	3.18	3.07	3.56	5.96	26.10
1.51-2.00	0.47	0.31	0.32	0.57	1.60	1.97	1.62	2.64	4.23	13.97
2.01-2.50	0.08	0.09	0.05	0.21	0.47	0.60	0.92	1.57	2.32	6.39
2.51-3.00	0.11	0.02	0.06	0.05	0.15	0.11	0.40	0.84	0.98	2.84
3.01-3.50	0.03		0.01		0.07	0.10	0.10	0.47	0.50	1.31
3.51-4.00	0.07				0.03	0.01	0.05	0.21	0.18	0.57
4.01-4.50	0.01						0.01	0.01	0.13	0.16
4.51-5.00								0.02	0.02	0.05
5.01-5.50										
5.51-6.00										
6.01-6.50										
6.51-7.00										
7.01-7.50										
TOTAL	8.81	5.88	3.75	5.73	10.83	9.52	10.28	15.42	22.73	100.01

Offshore Directed Swell: 7.06%

Table 2.4 (b) Adopted Directional Sea Wave Climate with NE 'seabreeze' Effect

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Spectral Peak Period (sec)									
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	TOTAL
0.00-0.50	3.39	1.53	3.24	2.04	3.02	2.64	3.64	2.77	2.23	34.05
0.51-1.00	1.57	1.91	4.16	1.62	2.51	2.84	2.97	2.40	1.69	30.76
1.01-1.50	0.82	0.57	3.00	0.31	0.81	1.03	1.49	1.27	0.52	11.94
1.51-2.00	0.46	0.16	1.49	0.17	0.73	1.41	1.76	1.16	0.68	8.90
2.01-2.50	0.23	0.09	0.66	0.03	0.54	1.20	1.84	1.61	0.70	7.21
2.51-3.00	0.05	0.03	0.06	0.03	0.19	0.78	1.16	1.04	0.35	3.75
3.01-3.50	0.01	0.01	0.02	0.00	0.16	0.23	0.47	0.52	0.29	1.78
3.51-4.00			0.02	0.01	0.02	0.22	0.19	0.26	0.13	0.87
4.01-4.50			0.01	0.01	0.03	0.10	0.16	0.11	0.06	0.49
4.51-5.00					0.02	0.05	0.06	0.01		0.14
5.01-5.50					0.01	0.01	0.02			0.05
5.51-6.00					0.01	0.00	0.01			0.02
6.01-6.50						0.00	0.00			
6.51-7.00						0.01	0.01			0.02
7.01-7.50						0.00	0.02			0.02
TOTAL	**6.53**	**4.30**	**12.68**	**4.23**	**8.07**	**10.51**	**13.81**	**11.16**	**6.64**	**0.04**

Offshore Directed Sea: 22.08%

Table 2.4(c) Adopted Directional Wave Climate - Sea plus Swell Using Peak Wave Train

Significant Wave Height (m)	Probability of Occurrence for Height and Period Shown Spectral Peak Period (sec)									
	N	NNE	NE	ENE	E	ESE	SE	SSE	S	TOTAL
0.00-0.50		0.01	0.01							0.03
0.51-1.00	1.11	0.90	0.40	0.52	0.97	0.81	0.95	0.95	1.53	10.46
1.01-1.50	3.16	2.46	1.52	2.81	4.11	3.39	3.83	4.66	5.90	35.58
1.51-2.00	1.26	0.98	0.75	1.06	3.16	3.86	3.77	4.11	5.76	25.87
2.01-2.50	0.43	0.22	0.21	0.35	1.36	2.23	2.83	3.41	3.66	15.00
2.51-3.00	0.18	0.08	0.11	0.14	0.38	0.98	1.64	2.04	1.57	7.33
3.01-3.50	0.05	0.01	0.03	0.01	0.24	0.33	0.59	1.07	0.89	3.32
3.51-4.00	0.07		0.02	0.01	0.06	0.23	0.23	0.48	0.31	1.44
4.01-4.50	0.01			0.02	0.03	0.10	0.18	0.13	0.18	0.66
4.51-5.00					0.02	0.05	0.06	0.03	0.03	0.19
5.01-5.50					0.01	0.01	0.02			0.05
5.51-6.00					0.01	0.00	0.01			0.02
6.01-6.50						0.00	0.00			0.00
6.51-7.00						0.01	0.01			0.02
7.01-7.50							0.02			0.02
TOTAL	**6.26**	**4.67**	**3.06**	**4.93**	**10.35**	**11.99**	**14.15**	**16.88**	**19.84**	**99.99**

Offshore Directed Peak Waves: 7.87%

Recorded Data - Height and Period

The recorded data for Brisbane has been split into sea and swell where a bi-modal spectrum exists, with the relevant wave heights based on the respective spectral energy. Slight modifications are then made for steepness (Ho/Lo = 0.02 as cutoff) and swell wave height losses due to diffraction. When the sea direction is from the west, a new sea is calculated and the corresponding swell height reduced respectively as the offshore propagating sea condition will not contribute to sediment transport.

Wave periods have been left unmodified.

Interpreted Data - Directions

Deepwater swell directions have been taken from BMO largely unmodified because of the lack of independent data for comparison and calibration. The directions attributed to sea waves are modified for significant offshore wind conditions. The sea directions are restricted to the range from north clockwise around to 180 degrees (easterlies) as more westerly winds will not contribute to sediment transport.

2.3.6 Adopted Directional Sea and Swell Wave Climate

The directional wave climate data adopted for use in this modelling investigation combines the recorded wave height and period results with the directions for sea and swell obtained from the BMO hindcasting. This required additional analysis of the recorded wave spectra for the Brisbane site to obtain the component sea and swell wave train significant heights and spectral peak periods.

This was undertaken jointly with coastal engineering staff of the Queensland Department of Environment. In-house computer routines were developed to automate this spectral splitting procedure as far as practicable, however it is recognised that the results obtained will contain some mis-matches between the recorded and hindcast sea and swell, particularly when more than two wave trains are present in the recorded data, or where the identified swell is decaying local sea.

The adopted non-directional wave climate is presented in Tables 2.3(a)-(c) in terms of the non-directional height/period probabilities of occurrence for swell, sea and combined waves with peak period respectively for the adopted 1990 to 1995 time series of data. Equivalent directional height/direction probabilities for those six years are presented in Tables 2.4(a)-(c).

2.3.7 Seasonal variability of Wave Conditions

There is significant seasonal variability in the wave climate in the Tweed/Gold Coast region. That variability is related strongly to the broad-scale weather patterns affecting the east coast of Australia, characterised broadly by:

- a well-defined cyclone season from December to April, with cyclones moving southward from the Coral Sea sometimes crossing the coast and sometimes passing well out to sea.
- sequences of high pressure systems to southern Australia in summer, generating persistent southeast to northeast winds and swell in offshore areas moderated by early morning land breezes and afternoon sea breezes which affect the locally generated 'sea' waves.
- a northward migration of the high pressure systems in winter, resulting in typically southwest to southeast winds and predominantly south to southeast swells in winter.
- occurrences of intense low pressure systems off the New South Wales coast in winter, generating moderate to high southerly swells.

The available directional wave climate has been analysed for seasonable variability with respect to both the local sea waves and incident swell. This has involved determining representative monthly mean significant wave heights and directions.

Both linear mean wave heights and (morphological) energy weighted mean wave heights have been derived. As well, the mean monthly wave directions are weighted towards the higher (morphological) wave energy components as follows:

Mean significant wave height $\dfrac{\sum H_S}{N}$

Weighted mean significant wave height = $\left\{\dfrac{\sum H_S^{2.5}}{N}\right\}^{0.4}$

Weighted mean wave direction = $\dfrac{\sum\left(H_S^{2.5} \cdot D\right)}{\sum H_S^{2.5}}$

where:

H_S is the significant wave height
N is the number of wave records
D is the wave direction between 0º and 180º

Monthly values of each of these parameters calculated for sea and swell are presented in Figure 2.2. This indicates:

- more or less uniform mean swell wave heights through the year, in the range 0.8 to 1.0 metres linear mean and 1.2 to 1.4 metres weighted mean.
- a significant shift in the mean swell direction from around east-southeast in summer to south to southeast during winter.
- a significant variation in sea wave heights through the year, being larger (up to 1.3 metres linear mean and about 2.0 metres weighted mean) in the summer cyclone season, and lower (typically 0.5 metres and 1.0 metres respectively) during winter.
- variable mean sea directions through the year in the range southeast to east-southeast.

  Review of the daily wave direction time series indicates greatest occurrence of northeast sector sea waves during spring (September to November) reducing through summer, with almost exclusively southeast sector waves in winter.

2.3.8 Long Term Variability of Wave Conditions

Previous reports, in particular the 1992 Delft Report (Roelvink and Murray 1992), have identified that the total wave energy can vary significantly from year to year and show long term trends. The total BMO hindcast directional data period available covers the period January 1989 to June 1996. While this is a significantly long period in terms of data availability, it is relatively short in terms of the variability of meteorological patterns, particularly relating to extreme events.

An analysis has been undertaken of the relative significance of the available directional wave data set used in this study in the longer term context. This has involved comparison of the non-directional wave energy levels against those from the longer recorded data set, and review of weather patterns and cyclone occurrences as noted over many decades.

Recorded Wave Energy Comparisons

Plots of the recorded Brisbane wave height percentage exceedances for the individual whole years 1989 to 1995, and for the average over that period compared with the average for the longer period 1977 to 1996 for Brisbane are presented in Figures 2.3 and 2.4 respectively. The percentage exceedance of wave heights for the period 1989 to 1996, used for this project, compared with the long term Brisbane records shows substantially higher than average wave heights, with 1989 and the first half of 1996 standing out as having unusually high wave energy.

Summary details in the form of plots of wave height occurrence, cyclone frequency and wave energy for an extended historical period are given in Figure 2.5.

The time series plot (Figure 2.5) of a function of wave energy related to sand transport capacity (taken as $Hsig^{2.5}$) shows that the period from 1977 to 1988 had relatively less energy than the period from 1988 to 1995. Another period of known high energy conditions is the time from 1967 to 1976, although no comprehensive wave data is available for this period. The plot of cyclone occurrences also shows a greater than average occurrence of higher waves during this time.

Analysis of the available recorded wave data clearly indicates that:

- the dataset used in this study, the years 1989 to 1996, contains an average wave energy 19% higher than the 19 year average.
- the single year 1989 contains transport capacity some 50% higher than the 19 year average.
- the year 1991 has relatively low wave energy, more or less similar to that for the years 1977 to 1987
- the period 1990 to 1995 is more representative of the longer term average, but nevertheless contains about 13% more sand transport capacity than the average.

It must be noted that the recording program has changed to hourly data recovery only in more recent years, and thus is now more likely to identify the most extreme conditions in each storm event. This may show up as somewhat higher wave energy levels overall, dominated by individual short term extreme events.

Long Term Weather Patterns

It is known that long term variations of weather patterns occur in which some years or groups of years contain greater or less extreme events or rainfall. Such variability may be closely linked to occurrences of El Nino events and may be indicated by the Southern Oscillation Index. It is considered that strongly positive SOI periods coincide with higher rainfall and probably increased cyclone occurrences and greater wave energy.

Figure 2.6 shows the SOI pattern over the period 1905 to 1995. A pattern of variation in which groups of years exhibit higher or lower values is immediately obvious, indicating some form of irregular cyclic behaviour. Of particular interest are the following features:

- The period 1954 to 1956 is known to have strong cyclone activity in southern Queensland, and shows high SOI values
- Similarly, years 1972, 1974 and 1976 were also cyclonically active and had high SOI values
- The years 1988 and 1989 are indicated with high SOI values and also show relatively high wave energy
- However, the period 1990 to 1995 show consistently low SOI values but have been assessed as exhibiting somewhat higher wave energy than the longer term (1977-1996) average

Wind data from Cape Moreton lighthouse has also been analysed for the period 1957 to 1996, as shown in Figure 2.7. This indicates the number of events per year of exceedance of 36 km/hr from the southeast sector. The known high wave energy years 1967 and 1972 again show up in this data, as does 1988 which appears to exceed 1989 in terms of this measure of wave energy.

Thus, it remains uncertain how the adopted period 1990 to 1995 fits with the overall long term pattern. Nevertheless, the single year 1989 clearly contains abnormally high wave energy and has been excluded from the assessment of average sand transport rates. The indications are that the adopted period may give

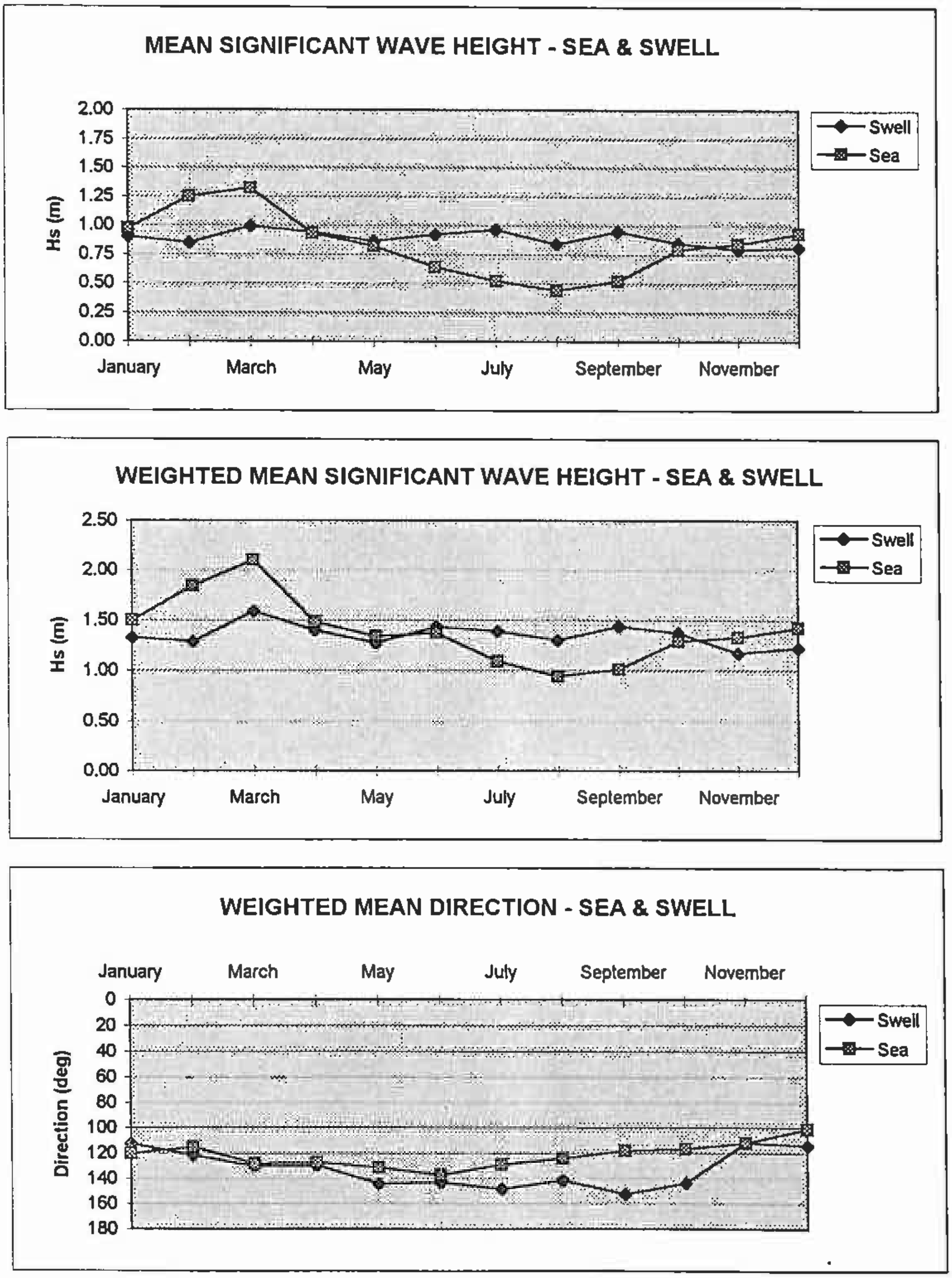


Figure 2.2
Monthly Mean Wave Conditions

Source :

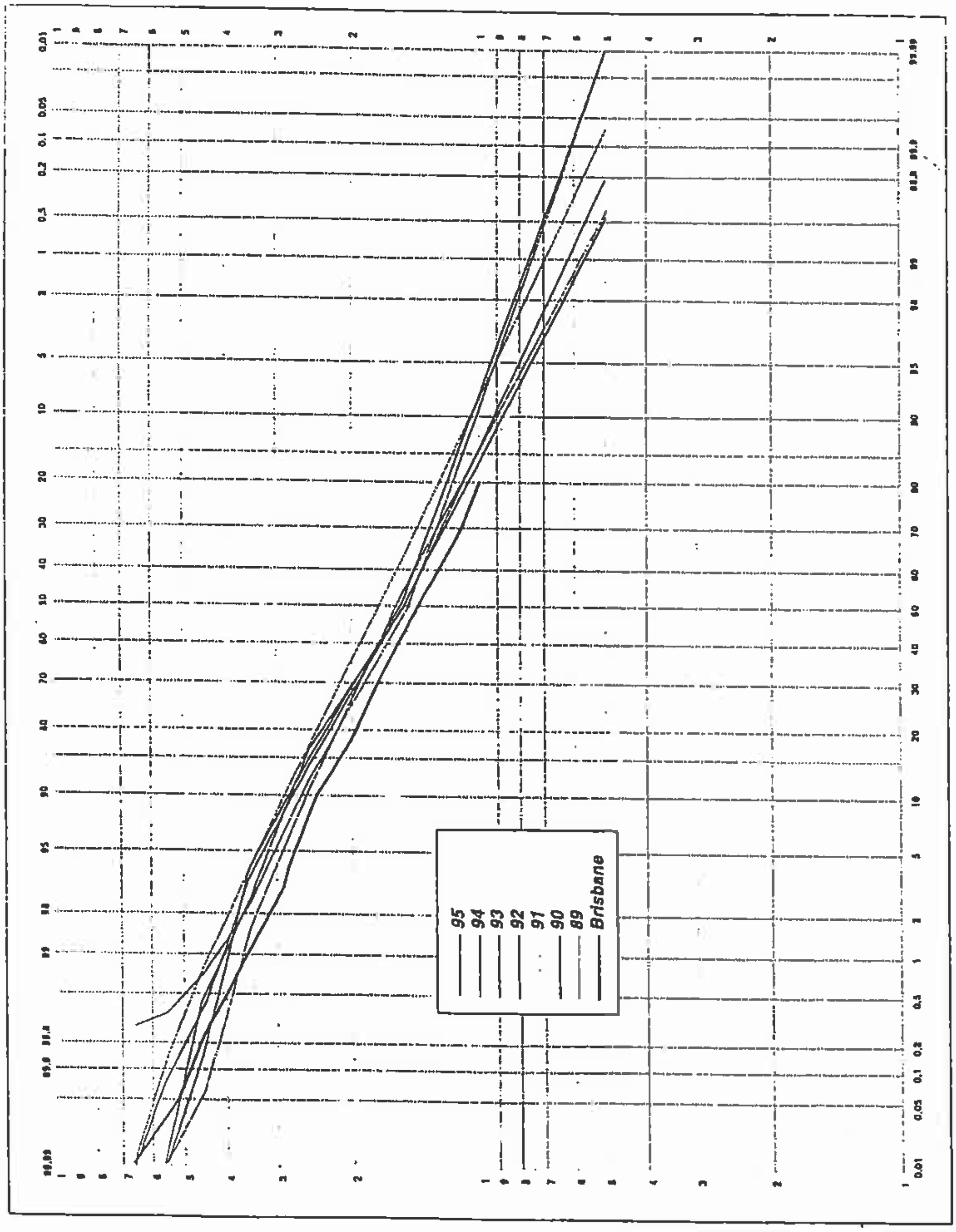


Figure 2.3
Wave Height Exceedances - 1989 to 1995

Source :

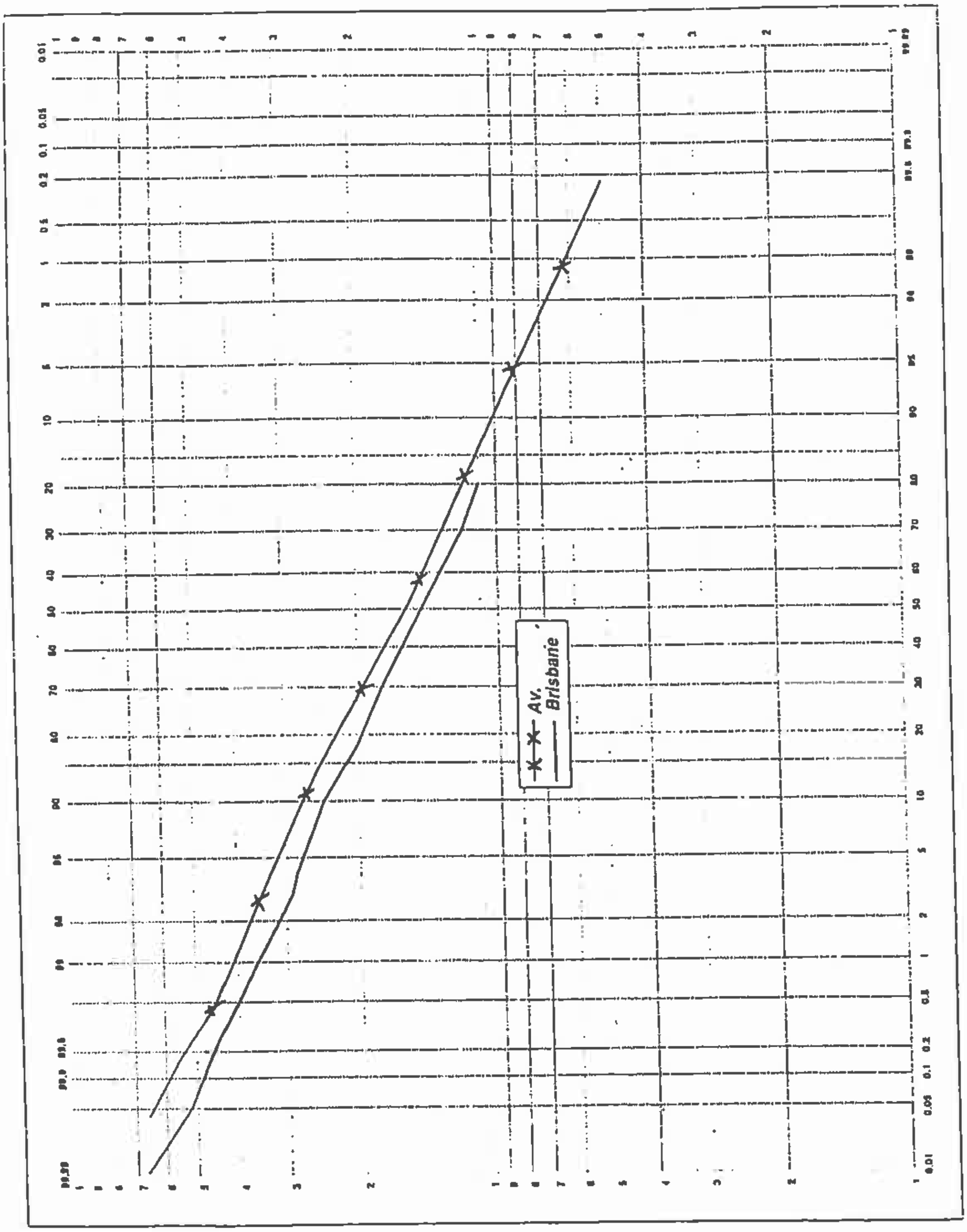


Figure 2.4
Wave Height Exceedances - 1989-1995 compared with 1977-1996

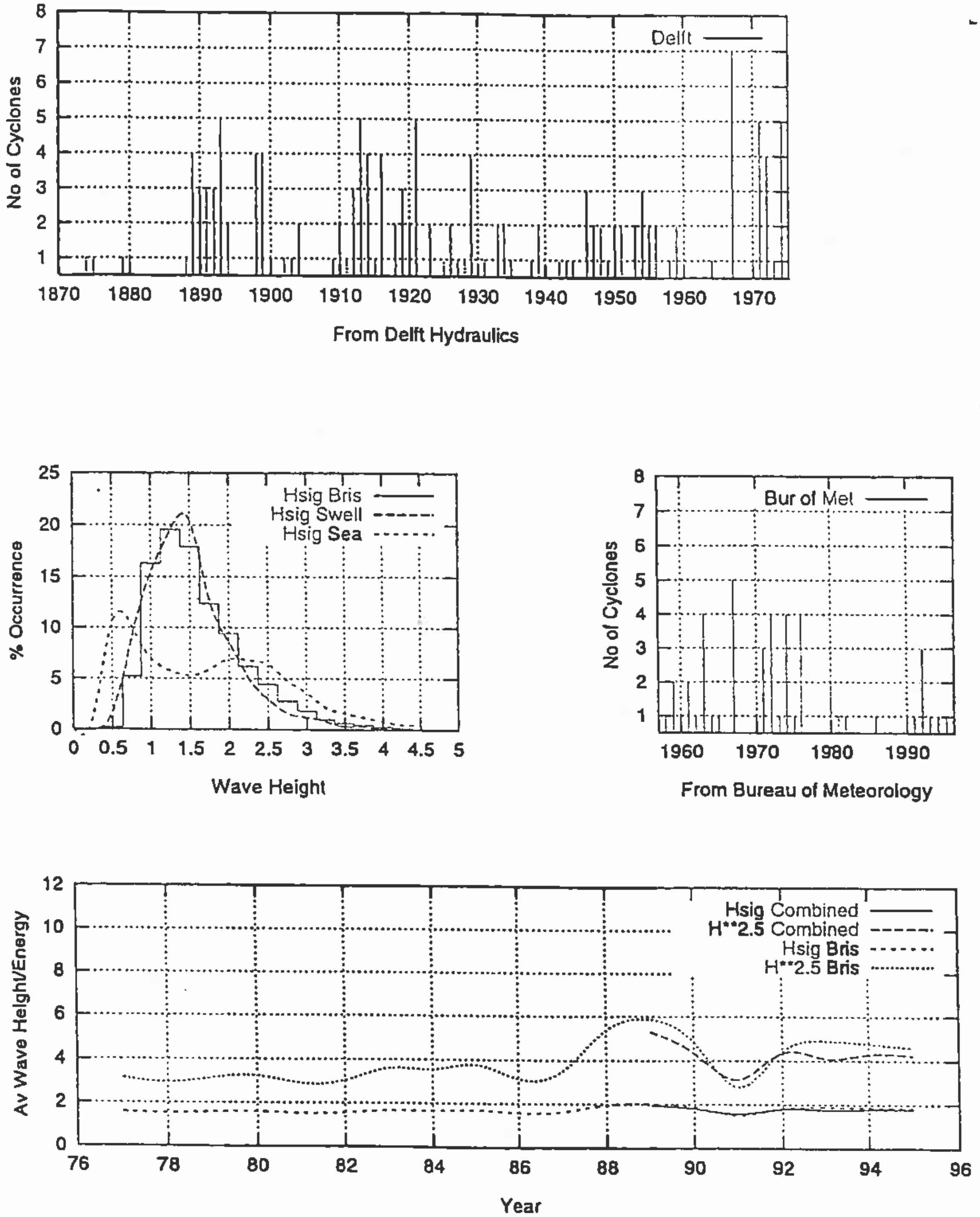


Figure 2.5
Long Term Wave Height Occurrence, Cyclone Frequency and Wave Energy Patterns

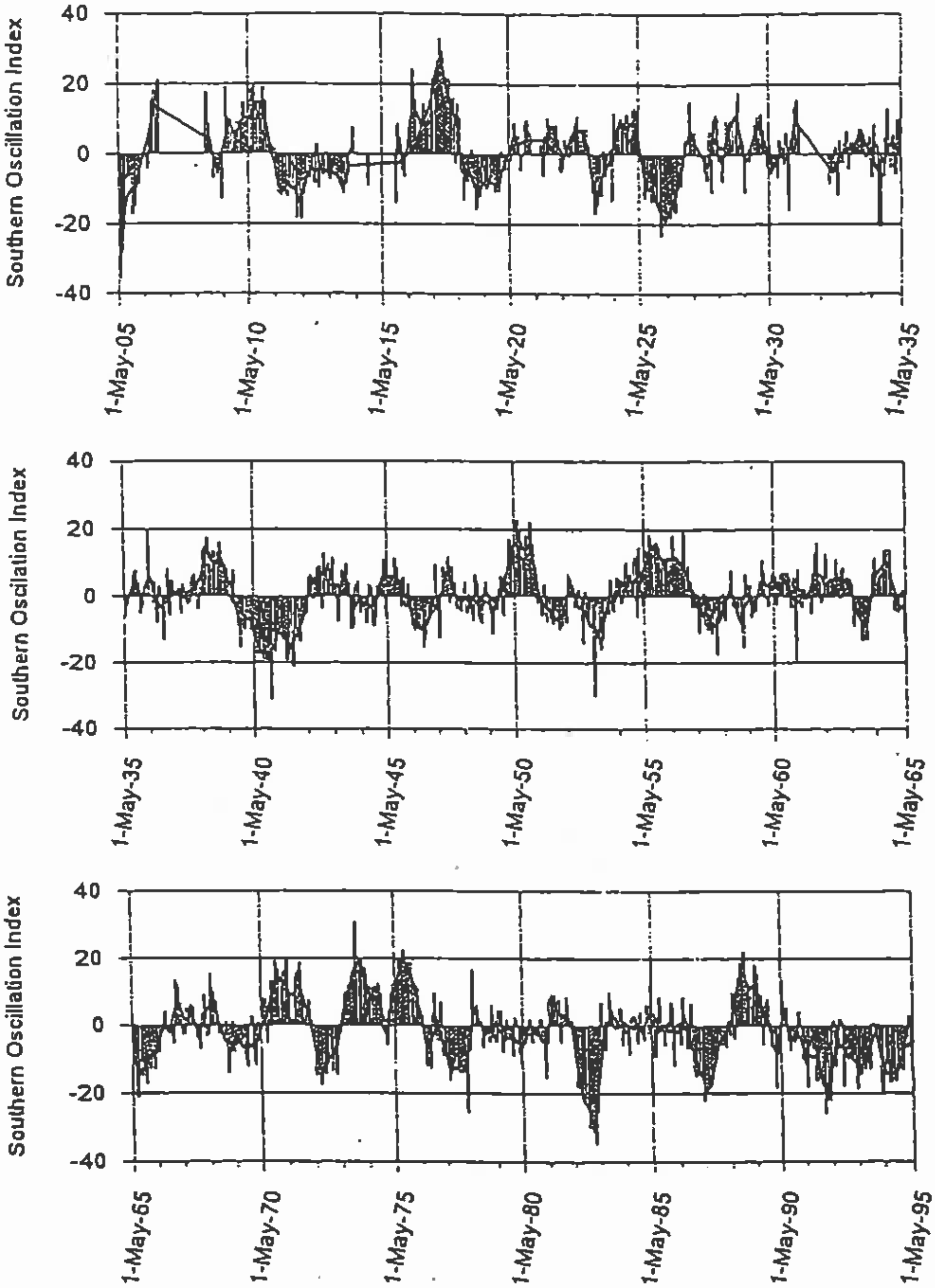


Figure 2.6
Southern Oscillation Index - 1905 to 1995

Source :

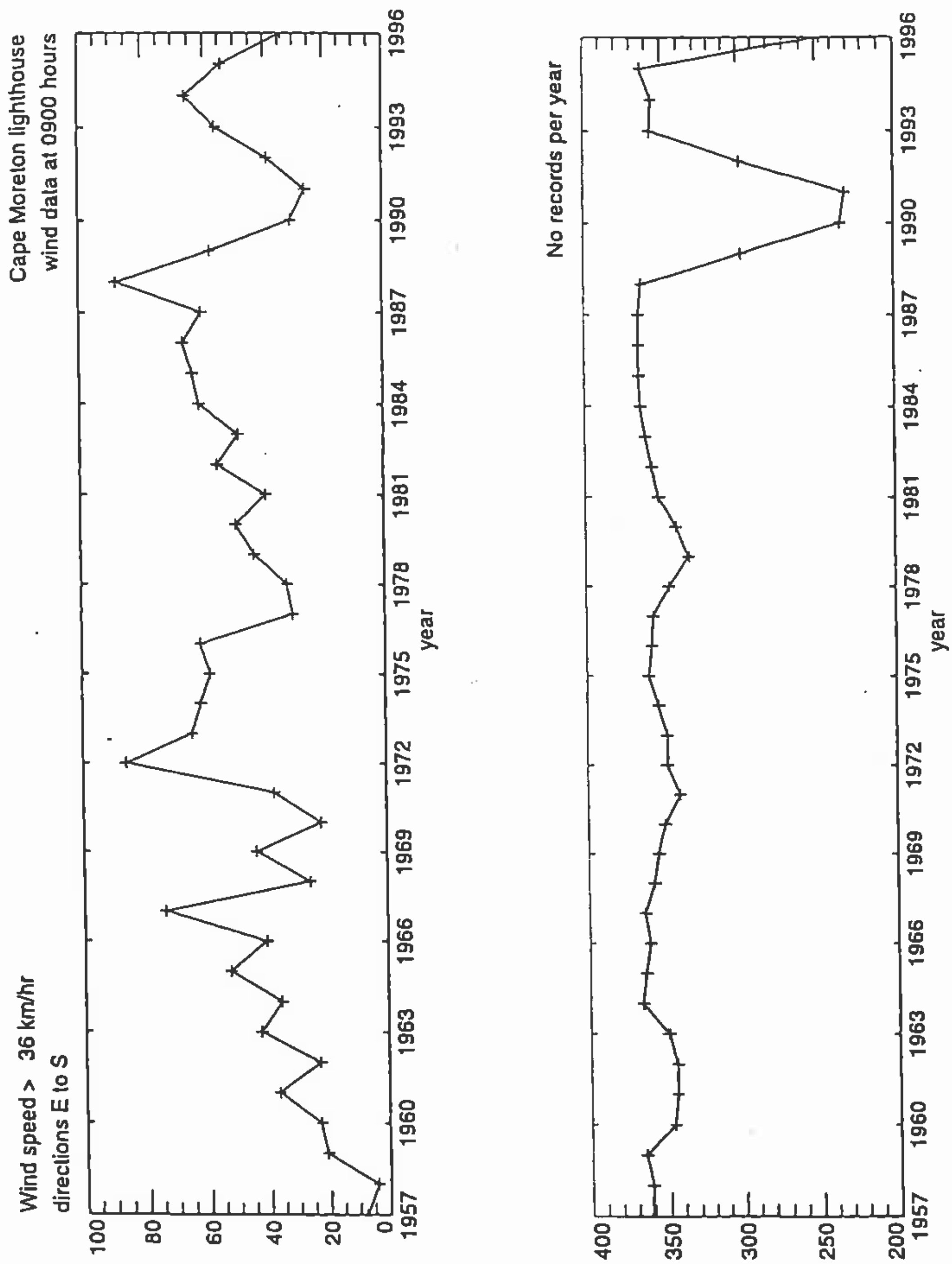


Figure 2.7
High Wind Speed Occurrences at Cape Moreton

Source :

somewhat higher than average transport (up to 13% higher). Certainly, it must be recognised that unusually high (1989) and low (1977-86 and 1991) wave energy years occur from time to time and need to be considered in the sand bypassing design and operation.

Extreme Wave Conditions

Extreme waves in the region are associated with cyclones and severe storms occurring as intense low pressure cells offshore. These are characterised typically with clockwise wind circulation around a central 'eye' which may remain offshore and move along the coast or make landfall at any location. The radius of high to maximum winds is typically limited to the range 20-150 kilometres, and the resulting wave conditions at any particular location are highly dependent on the track pattern of the eye.

Because of the clockwise wind rotation of these events, the dominant directions of the most extreme waves are in the east to southeast sectors. This is illustrated in Table 2.4(b).

Despite the considerable recorded data for the region, probabilities of occurrence of extreme wave conditions remain uncertain. This is due to the considerable variability of extreme wave occurrences over time frames which are long compared with the length of wave database. The directional characteristics of these waves are even less well defined.

Analyses to date indicate estimated extreme wave conditions as set out in Table 2.5. These have been derived from the wave height time series and exceedance probabilities for the Brisbane recorder for the period 1976 to 1994. Exceedance events were counted for each wave height analysed. This information was combined with the recorded exceedance (% time) data to provide estimates of mean durations of exceedance. A range of duration values was determined taking into consideration the uncertainty related to the fact that the earlier data is 6-12 hourly and probably did not detect the event peaks. This data was then combined with extrapolated average exceedance information to derive average recurrence intervals.

Table 2.5 Extreme Wave Conditions

Wave Height H_s (m)	Probability of Exceedance (%)	Average Recurrence Interval (years)
3.0	2.7	0.1
4.0	0.48	0.3
5.0	0.07	1.3-1.7
6.0	0.013	5-7
7.0	0.003	18-22
8.0	0.001	60-70

In the context of the proposed sand bypassing project which would continue in perpetuity, it is of significance for planning and design of the infrastructure and operational procedures to be aware of the probabilities of occurrence of extreme events within the timeframe involved. Table 2.5 illustrates the probabilities of occurrence of events with various average recurrence intervals over a range of operational design periods.

Table 2.6 Probability of Occurrence of Extreme Events

Design Period (years)	Probability of Occurrence During Design Period (%)						
	Average Recurrence Interval (years)						
	10	25	50	75	100	150	200
25	91.8	63.2	39.3	28.3	22.1	15.4	11.8
50	99.3	86.5	63.2	48.7	39.3	28.3	22.1
75	99.9	96.0	77.7	63.2	52.8	39.3	31.3
100	99.99	98.2	86.5	73.6	63.2	48.7	39.3
150	100.	99.8	95.0	86.5	77.7	63.2	52.8
200	100.	99.9	98.2	93.1	86.5	73.6	63.2

2.4 Wave Propagation

2.4.1 General Considerations and Methodology

Waves propagate from deep water to the nearshore areas of Letitia Spit, Point Danger, the southern Gold Coast beaches and the Tweed River through the processes of refraction, diffraction and attenuation due to bed friction. The latter is most relevant to the larger waves.

Analysis of wave propagation into the beach areas is fundamental to the determination of the sand transport regime at these areas. The southern Gold Coast beaches are substantially sheltered from the direct incidence of the predominant southeast sector waves. These beaches have a different wave exposure and hence a different sand transport regime from that occurring along the exposed beach and nearshore systems of Letitia Spit, Duranbah and Snapper Rocks.

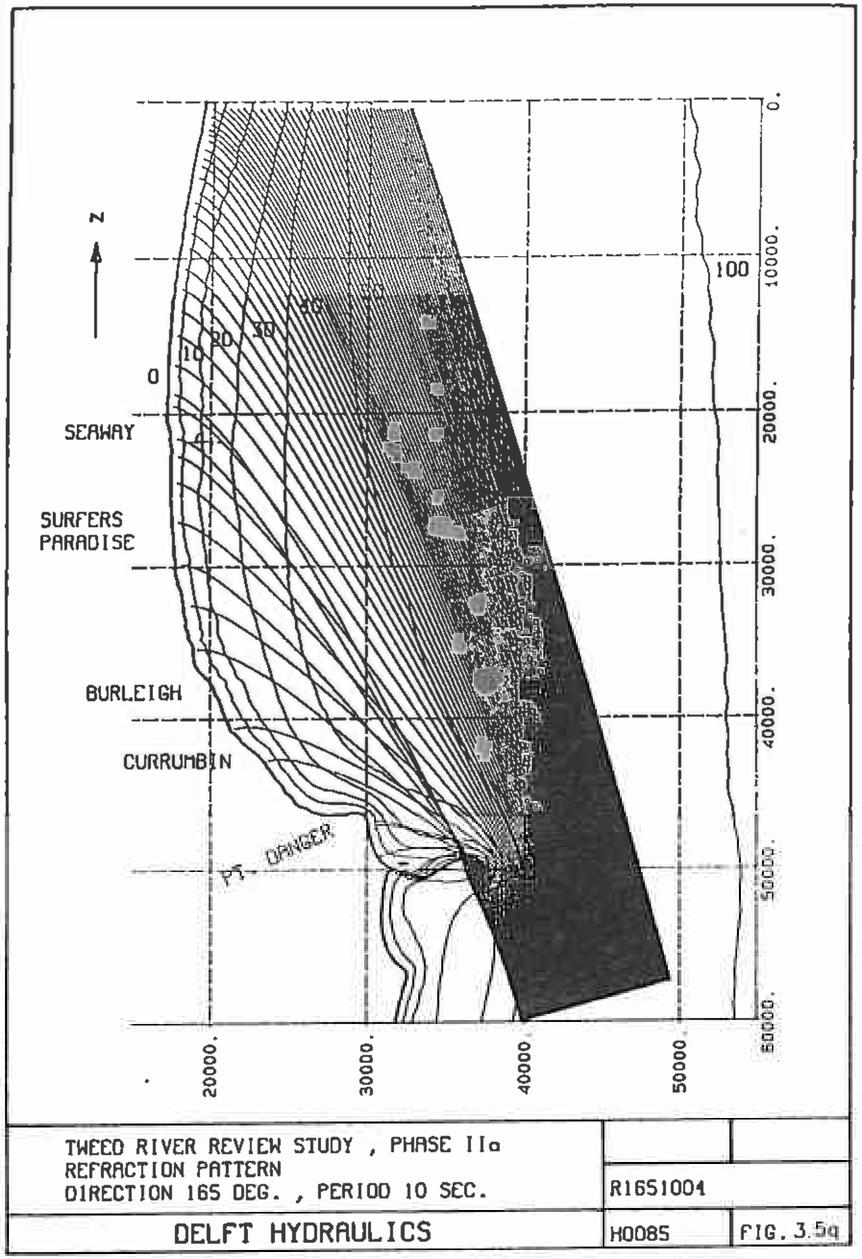
Conventional wave ray propagation analyses were undertaken by Delft Hydraulics (Roelvink and Murray 1992). Typical results of their analysis, reproduced herein for illustrative purposes as Figures 2.8 (a)-(c), show the general pattern of wave propagation to the various beaches. However, that analysis does not cater for the diffractive effects around Point Danger and will not give good results around the nearshore areas of Rainbow Bay, Greenmount and Kirra.

Wave propagation analyses for the present study have utilised the two-dimensional refraction/diffraction software system (RCPWAVE) developed for the US Army Corps of Engineers, and modified by WBM Oceanics Australia to more adequately cater for the particular requirements of this project. Specifically, these modifications include aspects to facilitate propagation past the prominent headlands of Point Danger, Greenmount and Kirra involving:

- diversification of the numerical solution scheme.
- utility software development (RCPR) to facilitate input file creation including model grid resolution refinement and distortion.
- nesting and rotation of sections of the model grid where required to overcome numerical solution problem areas near headlands and other obstructions.

The modelling system utilised thus caters as effectively as practicable for the combined processes of refraction, diffraction and bed friction.

Considerable effort was directed towards setting up the regional extent and appropriate boundary conditions to obtain representative results. In particular, both boundary selection and software modifications were trialed to overcome the inherent difficulties in modelling wave propagation over the reef areas seaward from Cook Island, based o the model extent and bathymetry shown in Figure 2.9.



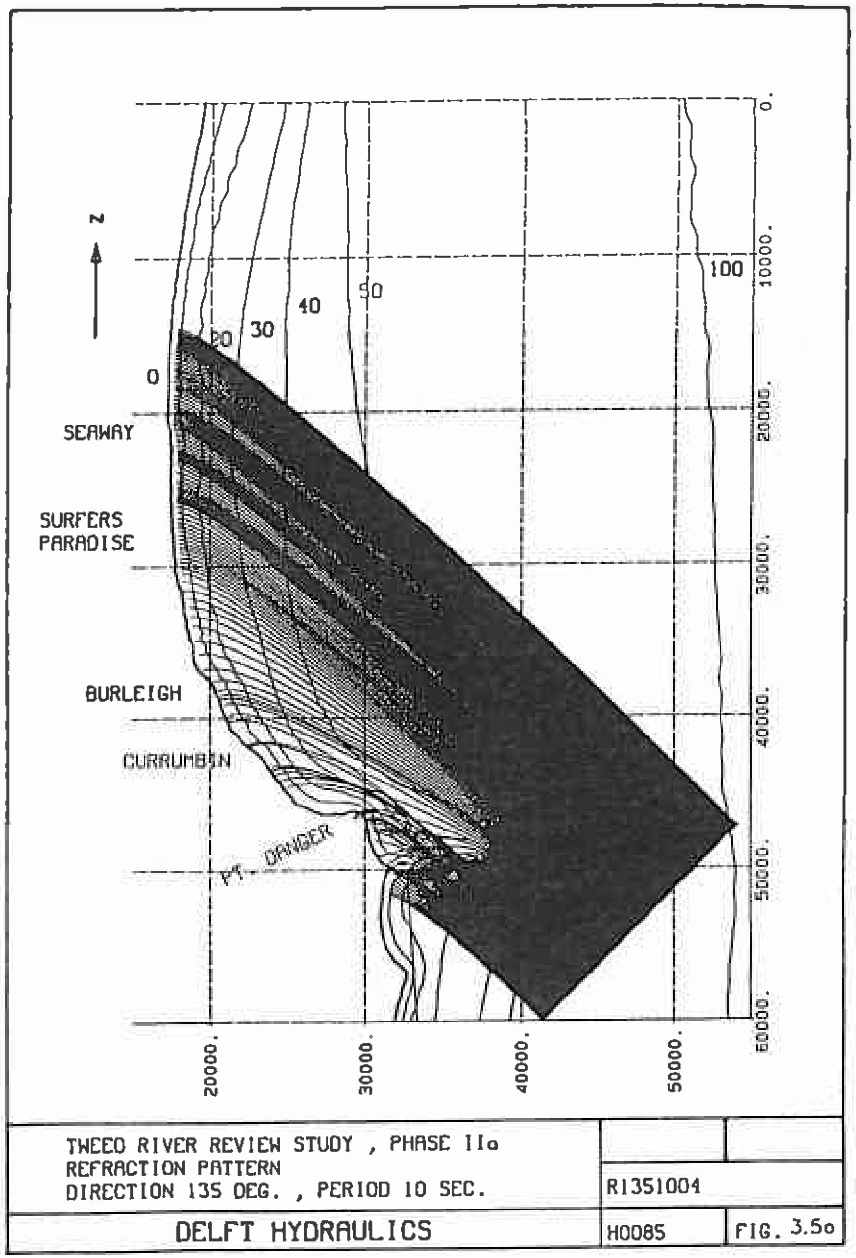
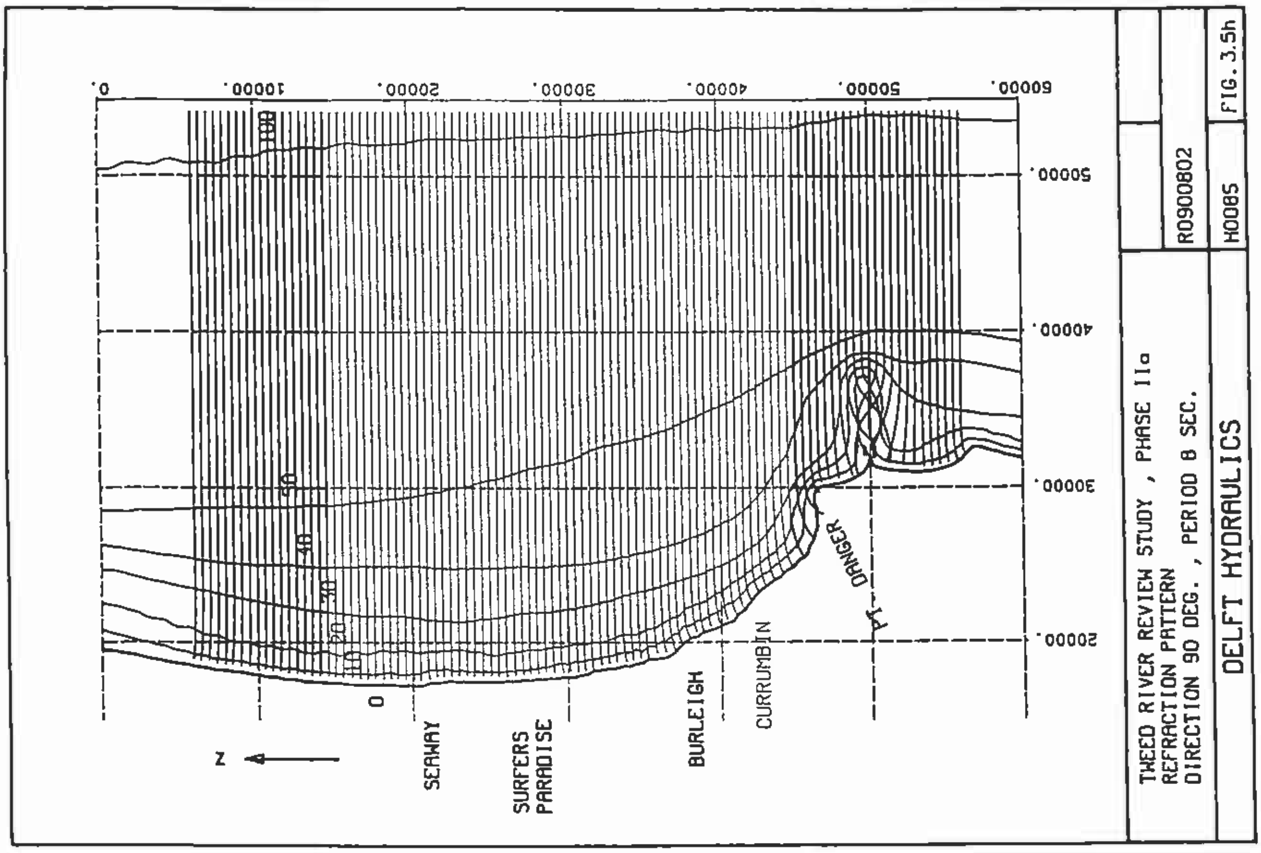


Figure 2.8a
Typical Wave Propagation Patterns - SSE and SE



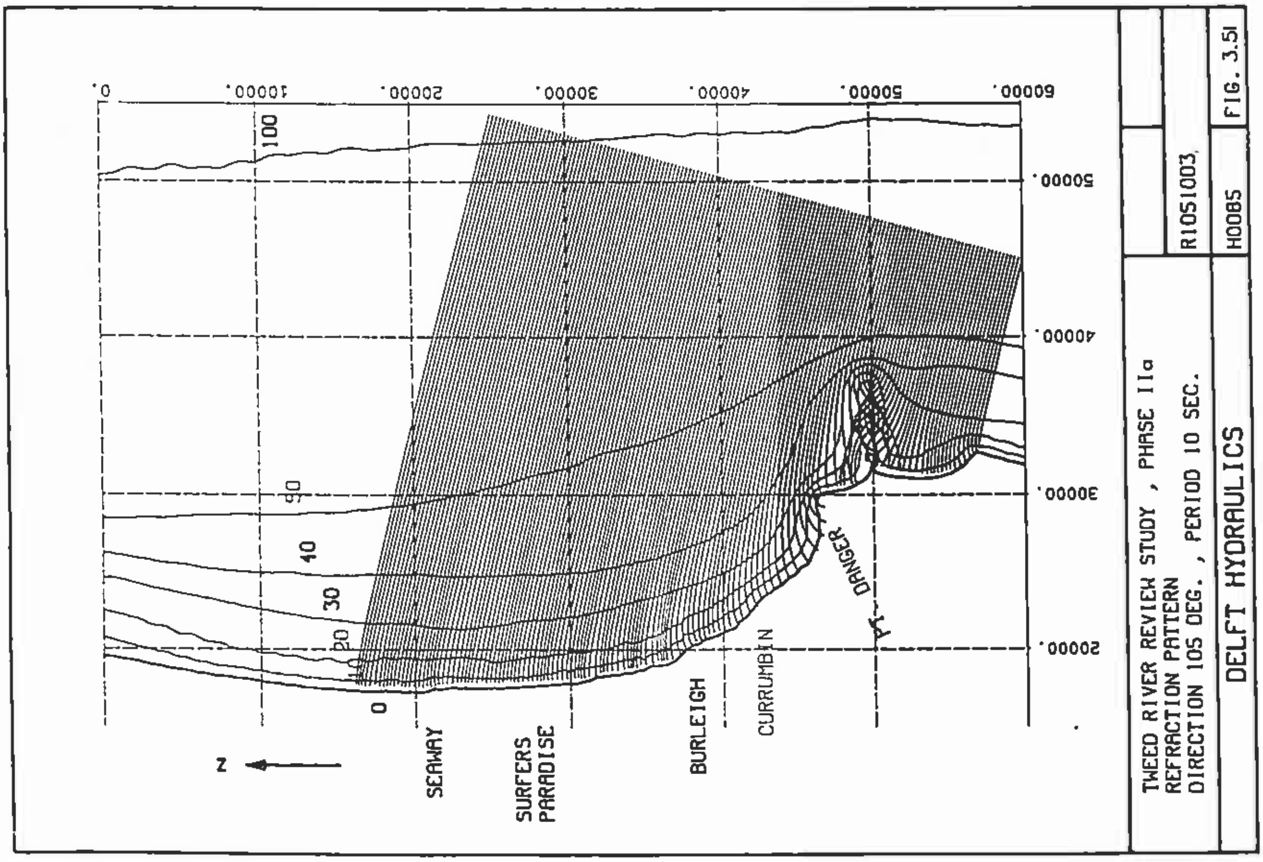
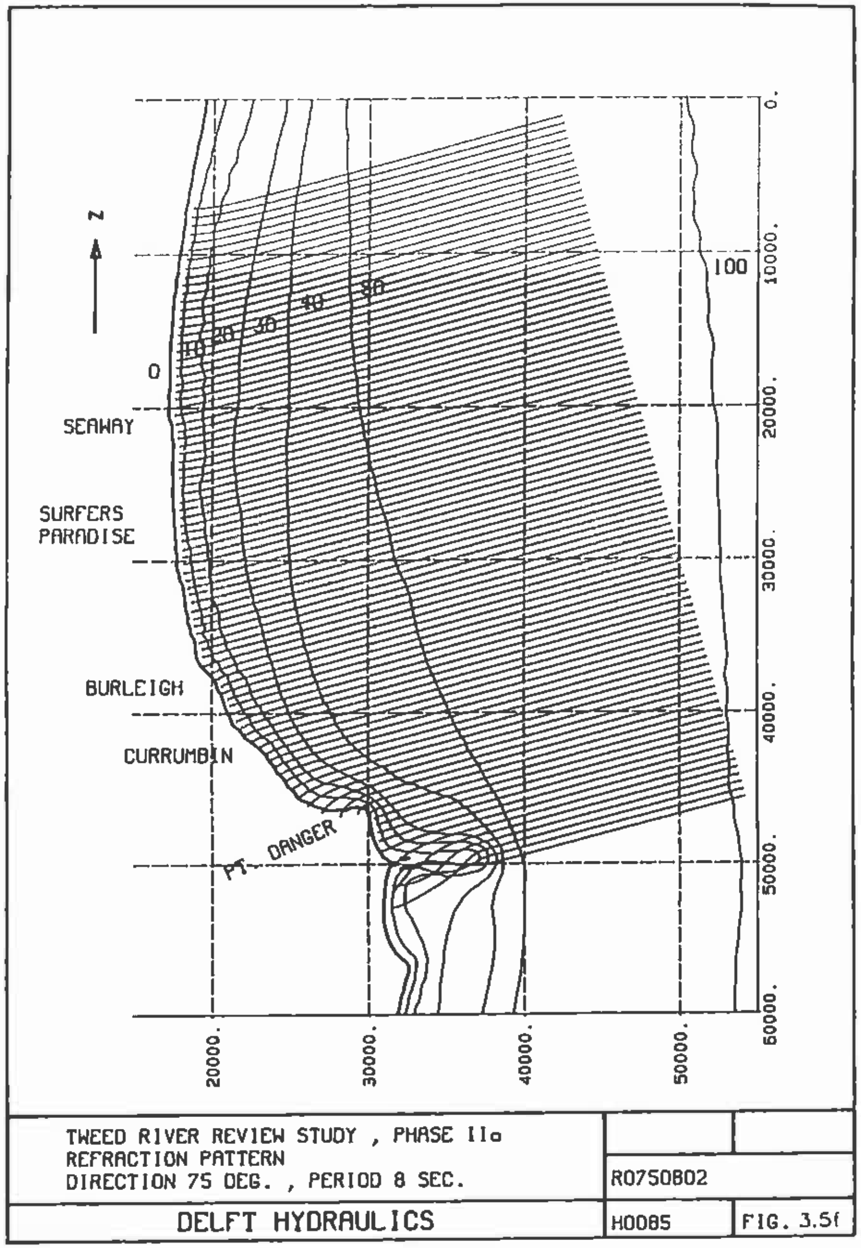
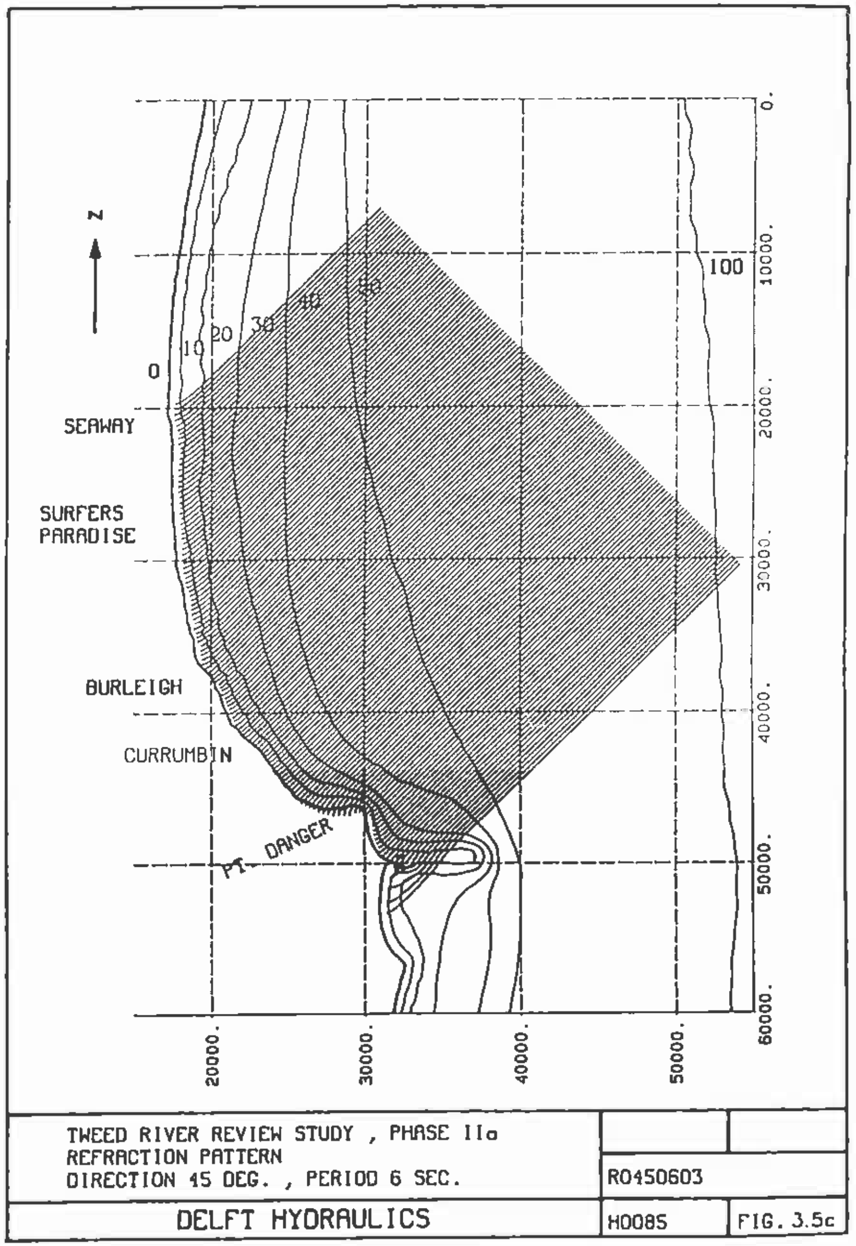


Figure 2.8b
Typical Wave Propagation Patterns - ESE and E

Figure 2.8c
Typical Wave Propagation Patterns - ENE and NE





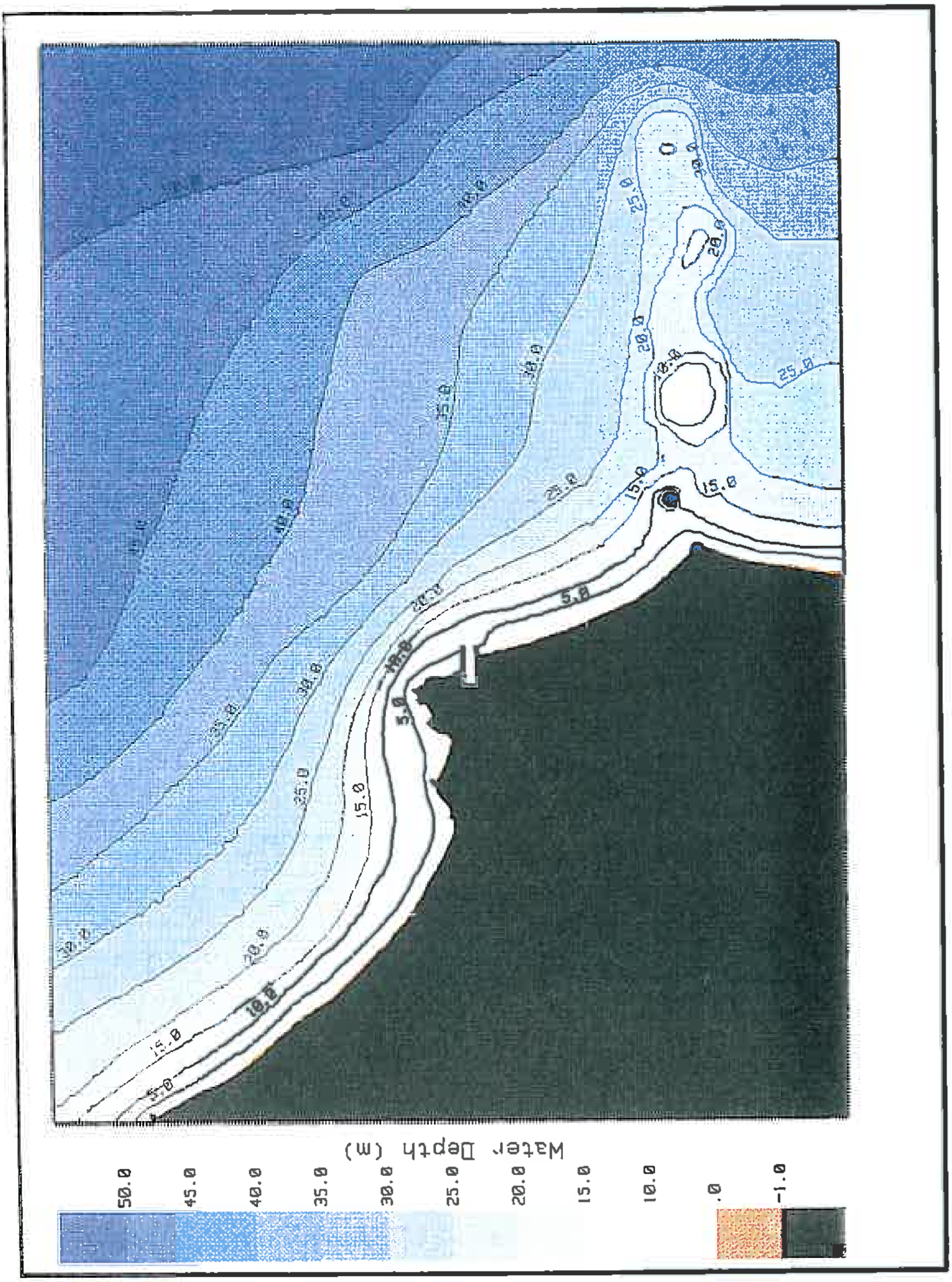


Figure 2.9
Wave Propagation Model Extent and Bathymetry

Wave ray propagation analyses as presented in Roelvink and Murray (1992) highlight the problems of analytical analysis of refraction in such complex areas, giving ray crossing and caustic formation patterns for waves from south to southeast. Equivalent problems occur with 2-dimensional modelling procedures where the real situation involves partial diffractive dispersion of energy along the wave crests together with separation of the waves into crossing wave orthoganols, as appears to be the case in this area.

It was found that the reef areas could not be modelled practicably, and that reasonable correlation of results with the measured data for the Tweed and Kirra recorder sites was obtained by setting the southern model boundary immediately adjacent to the northern side of the reef area. Thus the general bathymetric effects of the area were incorporated, but any focussing of wave energy by local shallow reef areas was not represented.

The analyses undertaken cover the full range of prevailing waves from the north-northeast through to the south-southeast. All other waves travelling from more westerly directions are of limited height nearshore and are of no significance to sand transport at the shore. The incident wave time series was determined such that this is properly accounted for.

2.4.2 Wave Propagation Model Results

Results of the model analyses of wave propagation from deep water to the Tweed and Kirra Waverider buoys have been compared with the best available assessment of the recorded height data and hindcast direction data as an indicator of model accuracy and any correction factoring which may be required. This validation procedure involved analysis of the deepwater (Brisbane and Byron), Tweed and Kirra recorded wave data to derive wave height factors ($H_{Inshore}/H_{Offshore}$) for recordings dominated by swell (Tp greater than 8 seconds) for the range of incident wave directions and direct comparison with the modelling results. The reliability of this analysis is limited because of the lack of recorded offshore wave directions.

The 2-D wave refraction/diffraction model showed a significant loss in energy of the swell waves at Kirra with respect to the deepwater waves coming from the southeast sector. To verify this result, two analyses were made.

Firstly a bulk comparison of all available swell data (TO2 > 6 secs) from all easterly directions was made by DoE. Those results indicated that, for northeast and southeast sector waves, the common occurrence of multiple wave trains which include both sea and swell in recorded data could be masking the true wave height relationship. Hence, the specific identifiable events for which the wave direction could be reliably estimated were used instead.

Three events where swell waves were consistently from the southeast for several days and one event where swell waves were consistently from the northeast for several days were individually analysed. Finally a combined plot of the Kirra/Brisbane wave height was produced and compared with the model results.

The wave propagation validation results are presented in Figure 2.10 and show the following:

- good correlation for waves from the east-northeast to east-southeast directions.
- slight over-estimation of wave heights from the northeast.
- apparent under-estimation of wave heights from the southeast to south-southeast.

As discussed above, uncertainty remains about the accuracy of the recorded reference data used, particularly in relation to multiple wave trains and lack of recorded offshore directions. Additionally, it is noted in the model results that there is a marked zone of lower wave height around North Kirra/Bilinga. The Kirra reference data is taken from a Waverider buoy located in the offshore section of this zone.

The nearshore bathymetry around the 10-15 metre depth zone off Greenmount to Kirra as represented in the model is somewhat different from that which has existed in the prototype in recent years, and this would have an effect of these results. It will take some time before natural redistribution of the sand placed nearshore in the recent dredging will reinstate the 1960's bathymetry as adopted in the modelling. Hence the model may better represent the longer term situation.

Hence, it is concluded that the model representation of wave propagation is sufficiently accurate for the present studies. Nevertheless, some corrective factoring may be needed for reliable determination of southeast swell wave conditions around North Kirra.

2.4.3 Impacts of Bypassing on Wave Propagation to the Beaches

Wave propagation into the beaches of Letitia Spit and the southern Gold Coast will be affected by the reduction of the entrance bar and the nearshore bathymetry offshore from Duranbah and the northern end of Letitia Spit. The extent of modification of the bathymetry and wave patterns will, in turn, affect the sand transport patterns in the area.

For the predominant prevailing southeast sector swell waves, the major impacts of the project will be as follows:

(i) Reduced effect of the entrance bar in refracting and focussing the waves at Duranbah. The southeast swell is expected to approach the beach more uniformly and with a somewhat greater angle (refer Figure 2.11).

(ii) Reduced effect of the tidal ebb jet and entrance bar in causing confused wave condition in the entrance channel immediately seaward of the training walls.

(iii) Reduced wave breaking in and near the entrance channel.

(iv) Reduced refraction of the more southerly waves, allowing more of the wave energy from those directions to impinge on the shoreline between Duranbah and Point Danger and at greater incident wave angle.

(v) Slightly increased wave heights in the entrance channel between the training walls.

For the smaller, shorter period sea waves, the effects will be less significant because of the lower potential for refraction. These waves will continue to propagate directly to the nearshore zone.

Wave propagation to the Gold Coast beaches north of Point Danger will be affected only slightly. Such effect relates only to those more southerly waves which would traverse over or near the river entrance bar or associated shoals in reaching the beach areas.

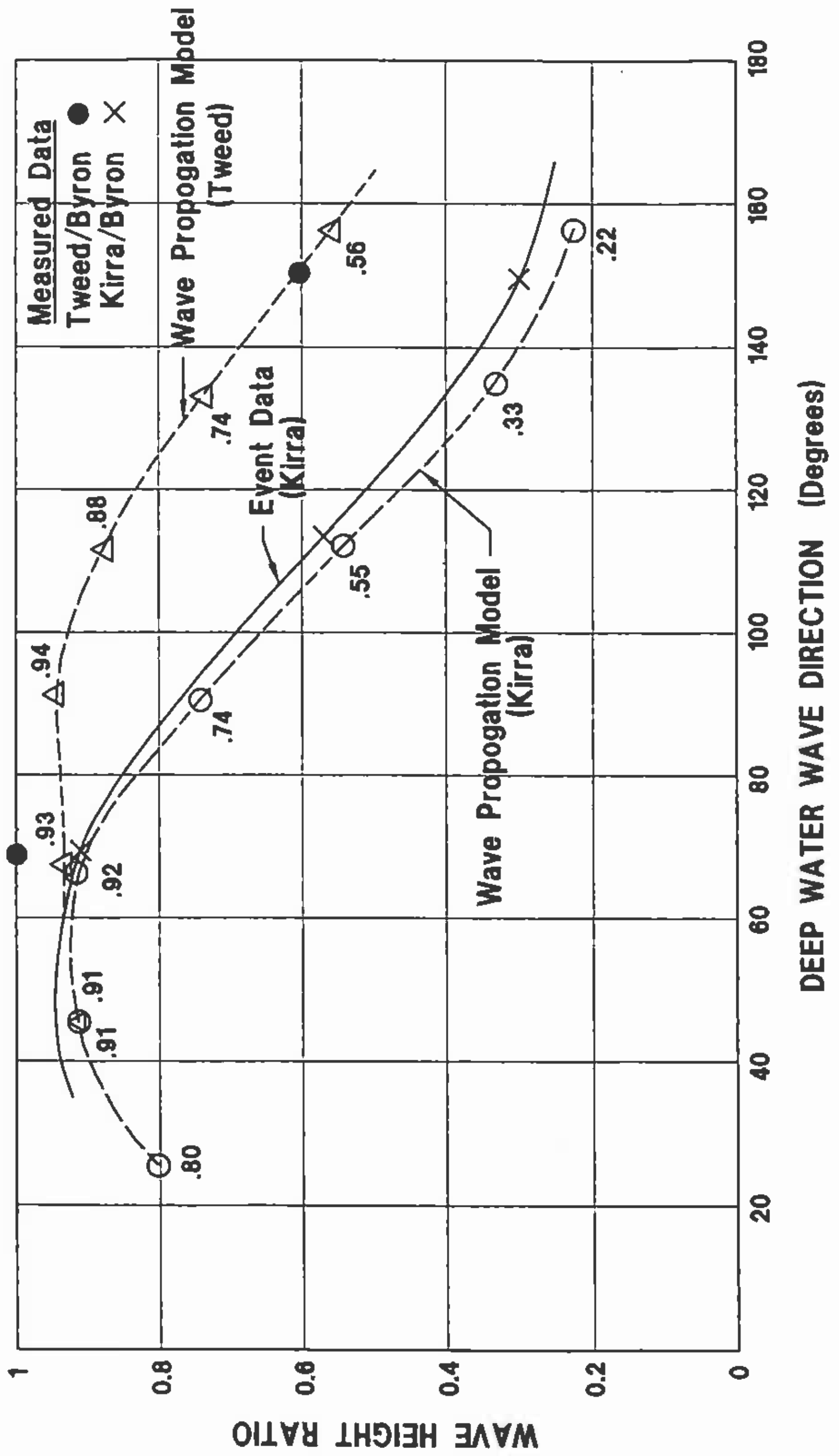
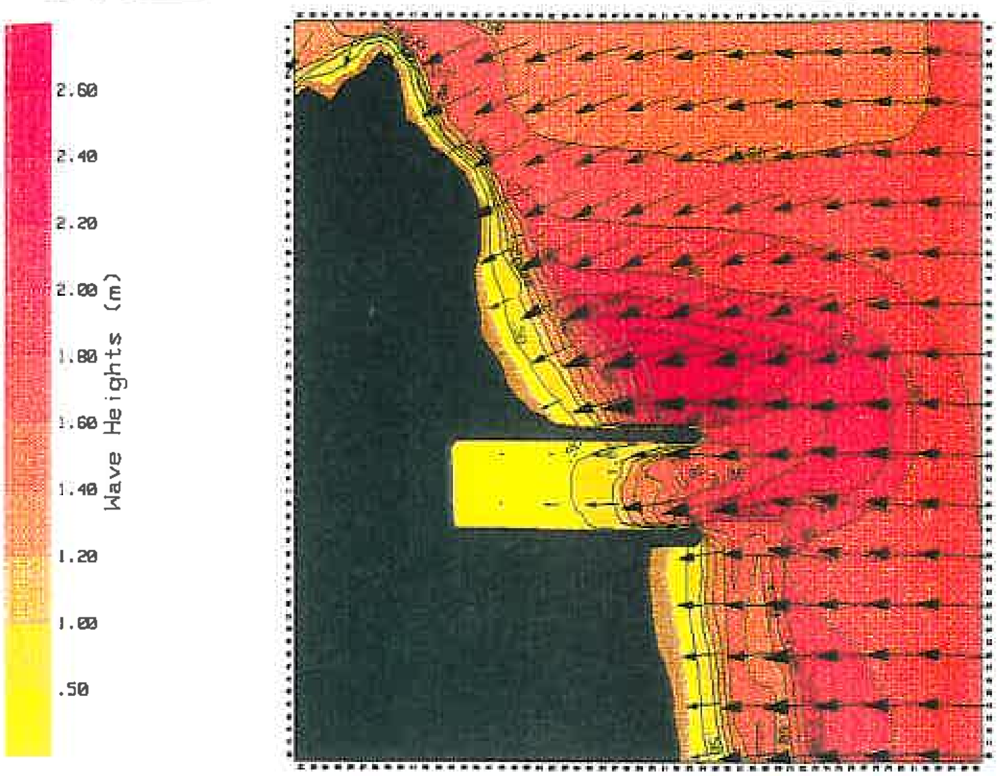
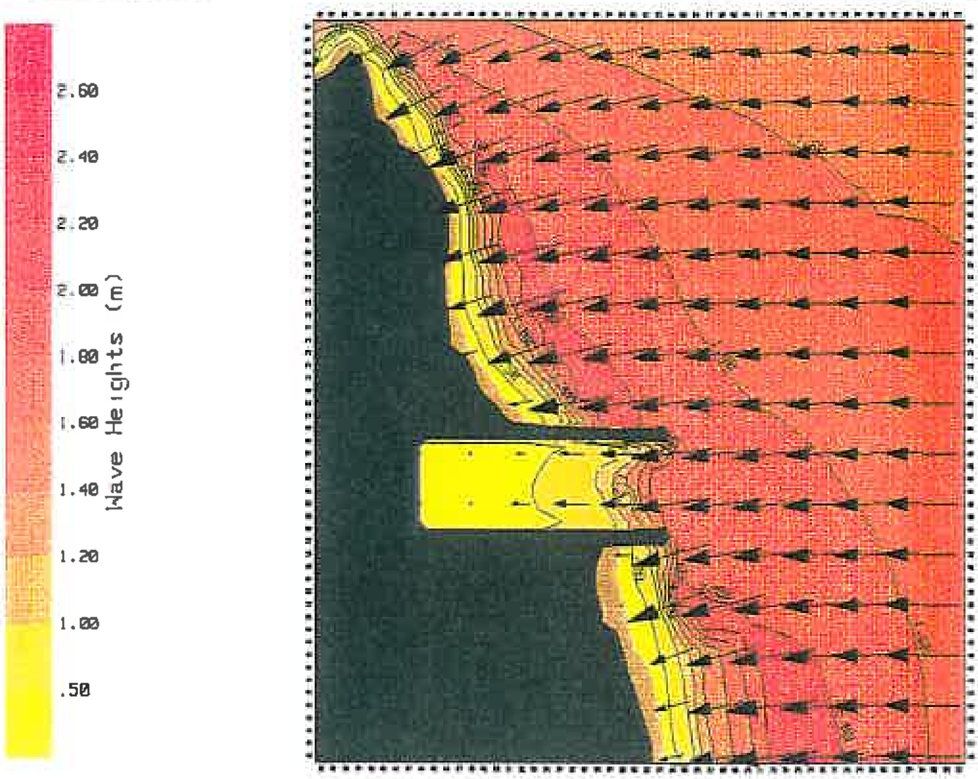


Figure 2.10
Wave Propagation Model Validation



WAVE PATTERN FOR PRE DREDGED ENTRANCE BAR BATHYMETRY



WAVE PATTERN FOR BYPASSING ENTRANCE BAR BATHYMETRY

Figure 2.11
Potential Bypassing Impact on Wave Refraction Near Tweed River Entrance

3 Nearshore Currents

3.1 General Considerations

Nearshore currents within and outside the surfzone and in the vicinity of the river entrance are important because of their substantial influence on sand transport and their significance for structural and operational design of the bypassing system.

Several factors contribute to generation of the general current patterns in the region, the most significant of which are:

- wave radiation stresses within the surfzone
- wind stress on the water surface
- tides
- the East Australian Current

Wave orbital velocities contribute locally to sand mobility and transport, and also must be considered in the design of structures subject to wave forces.

Each of these is discussed briefly below.

3.2 Wave Induced Longshore and Rip Currents

Longshore components of wave radiation stresses resulting from wave breaking in the surfzone generate longshore currents within and immediately outside the surfzone. Lateral 'escape' currents associated with wave set-up may also contribute to the longshore current in some areas.

The longshore current may occur in the gently sloping surfzone and swash area, or may occupy nearshore gutters or feed local rips through nearshore bars. Lateral mixing results in distribution of the longshore surfzone currents to the area immediately seaward of the breaker zone.

The speed of the longshore current depends on the breaking height and angle of the waves together with bathymetric features. Generally, longshore current speeds in the Tweed-Gold Coast area are up to about 1m/s, either upcoast or downcoast, in the zone out to depths of about 2-3 metres.

Greater current speeds up to about 1.5-2m/s may occur during storms and cyclones. At such times, the surfzone is wide (out to depths of 8 to 10 metres). The formation of nearshore bars during such events will tend to widen the zone of influence of the breaking waves and thus the zone occupied by the longshore current. There may be local increases in the longshore current adjacent to the headlands (Point Danger) and structures (groynes/breakwaters).

Results of the two-dimensional computer modelling of wave-induced nearshore currents are presented and discussed in Chapter 4 of this report.

3.3 Wind Induced Currents

Wind stresses on the water surface induce ocean currents. Near the coast these tend to become shore-parallel. Onshore winds may also create a vertical circulation in which the surface current is onshore with a compensatory seaward return flow near the bed.

No comprehensive data exists on wind generated currents in the Tweed region. Available data on currents indicates that the wind has a significant influence, coincident with other influences, on the nearshore currents.

A commonly adopted rule of thumb is that the wind generated current in the ocean is (order of) 1%-3% of the sustained wind speed. This varies significantly with water depth and the influence of the adjacent coastline. However, wind-induced currents are likely to have a significant influence on longshore sand transport at onshore wind speeds in excess of 15-20 knots.

It is likely that the wind typically generates nearshore currents of less than 0.2m/s, and that this may increase to about 0.5m/s during storms and cyclones. The wind induced current may be either increased locally or be divided into two broad current streams at Point Danger, depending on the wind and current direction.

The two-dimensional modelling undertaken for this study has included wind forcing associated with sea waves in the form of a superimposed wind on the sea surface equivalent to that required to generate the sea state being modelled. Results of the modelling are presented in Chapter 4.

3.4 Tidal Currents

The tidal wave at the Tweed-Gold Coast region propagates more or less from east to west, with little longshore components except near the north-facing southern Gold Coast beaches. The tidal currents are generally of low speed (less than 0.1m/s) except within the direct zone of influence of the Tweed River.

Current speeds through the Tweed River entrance are up to about 1.0-1.25m/s for a mean spring tide, and somewhat greater during large spring tides. The nature of the currents over the nearshore entrance bar depends on the bar bathymetry which changes continuously. Generally, the bar current pattern is:

- radial inflow over a wide directional spread on the flooding tide, and
- a concentrated ebb jet directed seaward on the ebb tide.

The ebb jet may be deflected to the north or south by the influence of winds, waves, the East Australia current or the local bar/channel bathymetry.

Results of computer modelling of the tidal current patterns in the Tweed River mouth region are presented and discussed in Chapter 7.

3.5 East Australian Current

The East Australian Current (EAC) has a significant direct effect on the nearshore current regime of the Tweed-Gold Coast area. A number of investigations have been undertaken to identify and quantity the nature of this effect (reference listing).

These studies have concluded the following with respect to the EAC:

- The EAC is directed downcoast (southward).

- The EAC generally flows along the edge of the continental shelf, though its position is highly variable, moving either east of the shelf or impinging on the coast.
- The headland at Point Danger may deflect the current in a local southeasterly direction immediately offshore from the headland.
- Such deflections may produce large scale clockwise recirculation cells in the coastal embayments to the immediate north and/or south of Point Danger.
- The circulatory current pattern of behaviour is complex and difficult to interpret quantitatively.

The EAC is most noticeable and may significantly affect sand transport at and offshore from Point Danger, the river mouth area and Letitia Spit. Recorded data from that area indicates it has speeds ranging up to 0.3m/s. It is less marked in inshore areas near Kirra and Bilinga and other beaches, although it has been observed at significant strength in water depths greater than about 7 metres there.

3.6 Wave Orbital Velocities

Wave orbital velocities may be computed directly from the prevailing wave conditions. Linear and/or cnoidal theories may be used as appropriate for a given wave condition and depth.

Orbital velocities up to 2m/s are relatively common at Gold Coast beaches. These may increase to about 3-3.5m/s during storms and cyclones.

Table 4.1 illustrates the range of orbital velocities for sea and swell waves in various water depths. Where these values exceed about 0.4-0.5m/s, there will be significant movement of the seabed sand.

Table 3.1 Wave Orbital Velocity at Seabed

Depth (m)	Peak Orbital Velocity (m/s)								
	Swell (Tp=8sec)			Sea (Tp=4*sqrt(Hs))					
	1	2	3	1	2	3	4	5	6
1	1.07	1.07*	1.07*	0.51*	0.74*	0.92*	1.07*	1.21*	1.33*
2	1.06	1.49*	1.49*	0.47	1.01*	1.27*	1.49*	1.68*	1.85*
4	0.72	1.44	2.16	0.27	0.93	1.82	2.02*	2.30*	2.55*
6	0.56	1.12	1.68	0.17	0.69	1.40	2.26	2.72*	3.03*
8	0.46	0.92	1.39	0.11	0.54	1.14	1.87	2.71	3.40
10	0.39	0.79	1.18	0.06	0.43	0.95	1.59	2.33	3.16
12	0.34	0.68	1.02	0.04	0.34	0.81	1.38	2.05	2.80
14	0.30	0.59	0.89	0.02	0.27	0.69	1.22	1.83	2.51
16	0.26	0.52	0.79	0.01	0.21	0.60	1.08	1.64	2.28
18	0.23	0.46	0.70	0.01	0.16	0.51	0.96	1.48	2.08
20	0.21	0.41	0.62	0.01	0.13	0.44	0.86	1.35	1.90

Note: designates breaking wave condition
designates zone of little or no sand movement

4 Longshore Sand Transport Modelling

4.1 General Considerations

4.1.1 Methodology

Sand is transported in the nearshore coastal zone by the complex interaction of waves and currents within the context of the shoreline shape and bathymetry. The prevailing waves may be locally generated sea or swell propagating from distant sources, and different sea and swell wave trains may coexist. The longshore sand transport modelling assessments undertaken for the present study incorporate these influences in a comprehensive manner. The general modelling approach is described below.

A time series of reliably recorded deep water wave spectra exists and has been split into component sea and swell wave trains with application of the hindcast sea and swell directions as described in Section 2. Two-dimensional (in plan) wave height and direction fields have been produced for each of a large range of incident waves. These include sea and swell over a range of heights and periods from seaward directions in 22.5º increments from north-northeast to south-southeast.

The 2-Dimensional modelling of current patterns and sand transport induced by wave radiation stress and wind forces has facilitated the determination of both the spatial and temporal variability of longshore sand transport. For each wave case modelled, the cross-shore distribution of longshore transport rates at each designated calculation site was extracted from the model output. Integration of the transport across each profile yields the total transport rate for each wave case.

Arrays of sand transport rates for both 'sea' and 'swell' were thus generated, in which the variability of transport as a function of wave height/period for 'sea' and wave period for 'swell' are established. Conventional theoretical relationships for swell-related transport are then applied.

The 6-hourly time series of sea and swell heights, periods and directions have been analysed to produced 6-hourly equivalent transport rates from interpolation and extrapolation of the array values. Daily, weekly, monthly and annual gross and net transport results are then derived from those results.

It has been noted in Section 2 that the available non-directional wave time-series for the period 1989 to 1996 may not be representative of the longer term average wave conditions. In particular, wave conditions in 1989 were abnormally (+50%) high in terms of the energy related sand transport capacity. Even ignoring that year of data, the wave energy over the years 1990-1995 appear to be somewhat higher (+13%) but more representative of the longer term average.

Hence, sand transport results averaged over the years 1990-1995 are regarded as indicative of the longer term average, pattern of behaviour, recognising that they may be somewhat higher than the longer term mean depending on the directional representativeness during that period. The results for 1989 and 1996 are considered as extreme in the context of the relatively short directional wave information available, but clearly are within the range of potential long term transport patterns for which the bypass system must be designed.

4.1.2 Uncertainties

The longshore sand transport processes within the study area are highly complex because of the complex nature of the shoreline, the nearshore bathymetry and the wave, current and wind interactions which occur there. Sand transport modelling of the type undertaken in this study is not undertaken routinely, and is subject to a range of inherent shortcomings related to the highly non-linear, time-dependent and spatially varying interactions involved.

It has been discussed, the wave climate data on which the modelling is based is limited in its duration and directional accuracy. This has significant consequences for the results particularly at Kirra.

As well, it is not feasible to provide for a range of secondary processes which, at some locations, may have a significant influence on wave, current and sand transport patterns. These include:

- wave refraction by local nearshore currents;
- time-varying nearshore seabed levels;
- wave reflections from rocky foreshores;
- interaction of micro-scale wave induced mass transport and larger scale longshore transport processes.

Of particular note is the constantly changing bathymetry of the nearshore bar/gutter system along the beach system. This has a significant influence on wave breaking and longshore current patterns which may vary substantially in the short term during storm erosion events. The modelling undertaken is based on adopted representative nearshore bathymetries. For Letitia Spit and Duranbah, two bathymetry situations have been modelled and an average results obtained.

Thus, the results of the modelling should be regarded as the best which can be obtained practicably, giving detailed insight into spatial and temporal variability patterns, while being subject to potentially significant uncertainty and error margins.

4.2 Sand Transport Formulation

Quantification of littoral drift is achieved by the use of two unifying and fundamental concepts:

(i) The wave orbital motion stirs up the bottom sands and puts it into oscillatory motion.

(ii) The bottom sediment, once mobilised by the waves, can be moved in the direction of the net current no matter how small that current is. These currents can be the net result of the oscillatory wave motion, unidirectional currents generated by the wave radiation stresses, or currents generated by other factors such as tides and wind.

Principal wave-generated currents are those which are generated within the nearshore zone by breaking waves. Such currents include alongshore currents, rip currents and the nearshore circulation systems associated with them. In the simplest form of wave generated current, waves breaking at an angle to the shore exert an alongshore thrust upon the water in the surf zone. This alongshore wave thrust provides the driving force for an alongshore current within the surf zone. The velocity distribution across the surf zone is determined for regular waves largely by lateral shear which causes the alongshore current to spread into the region immediately outside the break point, while for irregular wave the variation of breaker location caused by the wave height distribution contributes to this effect.

CERC Formula

Since obliquely approaching waves are usually the most important cause of the longshore current and the longshore sand transport, the most common simple model for sand transport relates the longshore transport directly to the wave properties. The wave energy flux (power per unit crest length) in deep water can be expressed as:

$$f\left(H_o^2, c_o\right)$$

where:

c_o = the wave speed in the deep water

H_o = the wave height in deep water

The energy flux entering the breaker zone per unit length of coastline can be expressed as:

$$f\left(H_o^2, c_o, K_{rbr}, \cos\phi_{br}\right)$$

where:

K_{rbr} = the refraction coefficient at the breaker line

ϕ_{br} = the angle of wave incidence at the breaker line

The component of this flux parallel to the coast can be expressed in terms of:

$$f\left(H_o^2, c_o, K_{rbr}, \cos\phi_{br}, \sin\phi_{br}\right)$$

The U.S Coastal Engineering Research Centre (CERC) studied a great quantity of model and prototype measurements and determined the following best-fit formula:

$$S = K_1 \times 10^6 H_o^2 c_o K_{rbr} \sin\phi_{br} \cos\phi_{br}$$

where:

H_o = the root mean square wave height

S = expressed as m^3/year

The value of the constant K_1 (commonly adopted in the range 0.44 - 0.7 x 10^6) is the subject of ongoing research. It applies generally but needs to be calibrated for fine to medium sands on moderately sloping beaches. It is believed that the value reduces for increasing sand grain size and where the nearshore extent of the beach sand zone is limited.

Wave/Current Relationships

Other more comprehensive formulae which incorporate the independent influences of longshore current and sand properties have also been developed. In the simplest form, the above CERC equation may be modified to:

$$S = 2.24 K_1 . H_b^2 \bar{v}$$

This may be used where the longshore current is strongly influenced by other factors in the nearshore zone and may be determined directly. A typical value for K_1 determined for Gold Coast conditions (Patterson, 1986) based on COPE observations, where H_b is the equivalent breaking significant wave height, is about 1200 where S is in m^3/day, or 0.41 x 10^6 from m^3/year.

Bijker Formula

The Bijker method (1968) incorporated the influence of the sand properties and seabed roughness into the wave/current type of formulation. It is summarised as follows:

S = Bed Load Transport + Suspended Load Transport
= $S_b + S_s$

where:

Bed Load: $$S_b = b.D.\frac{V}{C_o}.\sqrt{g}.e\left[-0.27\frac{\Delta D Co^2}{\mu.V^2\left\{1+\frac{1}{2}\left(\xi\frac{Uo}{V}\right)^2\right\}}\right]$$

in which: $\xi = 0.0575\ C_o$

$$C_o = 18\log\left(12\frac{d}{k}\right)$$

$$U_o = \frac{\pi H}{T}\frac{1}{\sinh\frac{2\pi d}{L}}$$

$$C_1 = 18\log\left(12\frac{d}{D90}\right)$$

notations:
S_b = bed-load transport in m^3/s per unit of width
b = factor (=5)
D = 50% grain diameter in m
D_i= i% grain diameter in m
V = current velocity in m/s
C_o = bottom roughness in m□/s
g = earth acceleration in m/s^2
Δ = relative density = $(\rho_s-\rho_w)/\rho_w$
μ= ripple coefficient = $(C_o/C_1)^{3/2}$
C_1 = bottom roughness due to grains alone
U_o = maximum orbital velocity above bottom in m/s
d = water depth in m
k = factor for ripples = □ (ripple height) in m
H = wave height in m
T = wave period in seconds
L = wave length in m

Suspended Load: $$S_s = I.83.S_b\left[I_1 In\frac{33d}{K}+I_2\right]$$

in which: $$I_1 = 0.216\frac{(\frac{k}{d})^{z-1}}{\left(1-\frac{k}{d}\right)^z}\int_{\frac{k}{d}}^{1}\left(\frac{1-y}{y}\right)^z dy$$

$$I_2 = 0.216\frac{(\frac{k}{d})^{z-1}}{\left(\frac{1-k}{d}\right)^z}\int_{\frac{k}{c}}^{1}\left(\frac{1-y}{y}\right)^z In\,y\,dy$$

$$z = \frac{w}{\beta K V_*^1}$$

$$V_*1 = \frac{V}{C_o}\sqrt{g}.\left\{1 + ½\,\xi\left(\frac{U_o}{V}\right)^2\right\}^{½}$$

notations: S_s = suspended-load transport in m^3/s per unit of width
w = fall velocity of 50% grain diameter in m/s
K = kappa, factor of Von Karman = 0.4
V_*^1 = shear velocity in respect to waves and current in m/s
I_1 and I_2 = integrals
β = a factor depending on the suspended sediment concentration and the intensity of turbulence.

Van Rijn Formulation

Van Rijn (1990) addressed the issue that reliable models to predict the time-averaged concentration profile for a rippled bed or a plane sheet flow bed were lacking. He proposed a new method based on the convection-diffusion equation and separate current-related and wave-related mixing coefficients. This involved introduction of separate current-related and wave-related bed roughness values. The method was developed to apply for non-breaking or breaking waves over rippled or plane seabeds.

His relationship has the following form:

$S = S_b + S_s$

where:

bed load transport $$q_{b,c} = 0.25\, u'_{*,c}\, d_{50} \frac{T^{1.5}}{D_*^{0.3}}$$

$$u'_{*,c} = \left[\bar{\tau}'_c / \rho\right]^{0.5}$$

$$T = \left(\bar{\tau}'_{cw} - \bar{\tau}'_{cr}\right) / \bar{\tau}_{cr}$$

$$D_* = d_{50}\left[(s-1)g / v^2\right]^{1/3}$$

bed-shear stress by current: $$\bar{\tau}'_c = \frac{1}{8}\rho\, \alpha_{cw}\, \mu_c\, f_a\left(\bar{V}_R\right)^2$$

bed-shear stress by waves: $$\bar{\tau}'_w = \frac{1}{4}\rho\, \mu_w\, f_w\left(\hat{U}_\delta\right)^2$$

bed-shear stress by current/waves: $$\bar{\tau}'_{cw} = \bar{\tau}'_c + \bar{\tau}'_w$$

wave orbital velocity: $\hat{U}_\delta$

uniform current velocity : $\bar{V}_R$

efficiency factor current: $\mu_c = f_c' / f_c$

efficiency factor waves: $\mu_w = 0.6 / D_*$

wave-current interaction coefficient:

$$\alpha_{cw} = \frac{\ln^2(90\delta_w / k_a)}{\ln^2(90\delta_w / k_{s,c})}$$

$f_{c'}$ = current related friction factor from k_{sc}

f_c = grain size related friction factor

f_w = wave related friction factor from k_{sw}

f_a = friction factor derived from k_a

k_a = apparent bed roughness

= $k_{sc} \exp[\gamma \hat{U} / \overline{V}_r]$

bed concentration: $$c_a = 0.015 \frac{d_{50}}{a} \frac{T^{1.5}}{D_*^{0.3}}$$

suspended load transport (numerical integration):

$$q_{s,c} = \int_a^h u c \, dz$$

This integral may be approximated by the alternate formulation as follows:

suspended load transport: $$q_{s,c} = (F_c + F_w)\overline{V}_R h c_a$$

current-related correction factor: $$F_c = \frac{[a/h]^{ZC} - [a/h]^{1.2}}{[1.2 - ZC][1-(a/h)]^{ZC}}$$

wave-related correction factor: $$F_w = \frac{[a/h]^{ZW} - [a/h]^{1.2}}{[1.2 - ZW][1-(a/h)]^{ZW}}$$

current-related suspension number: $$ZC = \frac{w_s}{\beta \kappa u_{*,c}}$$

wave-related suspension number: $$ZW = \alpha[\frac{w_s}{\bar{v}_R}]^{0.9}[\frac{\bar{v}_R T_p}{H_s}]^{1.05}$$

$\alpha = 7$ for $h \geq 100\delta_s$

$\alpha = 0.7(h/\delta_s)^{0.5}$ for $h < 100\delta_s$

The reader is referred to the Van Rijn (1990) reference for a description of all of the parameters involved in this formulation. For present purposes, it is sufficient to emphasise that the results of the method are strongly influenced by the bed roughness, reference level and near bed mixing layer thickness values. In particular, they depend intimately on how those parameter values are used in combination.

Van Rijn offers the following advice on selection of these values.

Bed roughness $k_{s,c}$, $k_{s,w}$: A reasonable estimate for currents and non-breaking waves is $k_{s,c} = k_{s,w} \approx 3\Delta_r$, with values in the range of 0.03 to 0.1m. In case of breaking waves with sheet flow conditions the bed roughness will be of the order of the wave boundary layer thickness giving $k_{s,w} \approx \delta_w$ with values in the range of 0.01 to 0.02m.

Reference level a: The reference level is proposed to be equal to half the rippled height $(a=\square\Delta_r)$ in the case of non-breaking waves and equal to the wave boundary layer thickness $(a=\delta_w)$ in the case of sheetflow conditions.

Near-bed mixing layer thickness δ_s: This parameter can be obtained from a relationship given in the reference, giving $\delta_s \approx 3\Delta_r$ in the ripple regime and $\delta_s = 3\delta_w$ in the sheet flow regime. Both expressions yield values in the range of 0.03 to 0.1m. In the case of breaking waves the δ_s value may be somewhat larger ($\delta_s \approx$ 0.2m) due to the breaking effect. More field data from the surf zone are necessary to better define the δ_s parameter for breaking wave conditions.

Preliminary comparison of Bijker and Van Rijn methods indicates the following:

(i) Van Rijn requires setting separate bed roughness values relating to current and waves, and the result is quite sensitive to these values. Furthermore, the wave-related bed roughness outside the surfzone may be different from that in the surfzone.

(ii) Van Rijn appears highly sensitive (highly non-linear) to the effect of increasing the uniform current velocity, probably being the main reason this method tends to over-estimate transport rates. Despite that, it may under-estimate transport for low uniform current speeds (even with somewhat higher waves).

(iii) Bijker is a more stable method varying more linearly with both current and wave height, but probably under-estimates the effects of the higher waves and uniform current. It thus gives low values for the bigger storm events.

(iv) Bijker gives relatively high values in deeper water. This is due more to the 'z' exponent in the suspended load than to the 'b' factor which is often arbitrarily reduced outside the surfzone. It also reflects the lack of mobility threshold in its formulation.

(v) The method used and the way it is applied affect the results of the cross-shore distribution calculations.

We have adopted the Van Rijn formulation for this project, given that it is the more recent generally accepted method which draws upon and attempts to improve the other available methods. Nevertheless, our experience is that this method is subject to overestimation of sand transport rates where the uniform current speed is high, as in the surfzone, particularly adjacent to the headlands, for storm conditions. Further investigation and validation for extreme prototype conditions is needed to obtain good results for such situations.

4.3 Two Dimensional Modelling Software

Conventional 'one-dimensional' longshore transport calculation using a wave/current formulation (eg. Bijker or Van Rijn) generally involves assessment of the longshore current by a standard method involving bed slope, breaking wave height and breaker angle. The cross-shore distribution of longshore current is usually estimated from a standardised shape depending on an adopted lateral mixing parameter.

This approach is acceptable for simple long beach units but is not appropriate for the present study area with headlands, the river mouth and abrupt changes in shoreline orientation. Hence a 'two-dimensional' modelling approach has been necessary, in which:

(i) the longshore current pattens are determined from the fundamental forcing factors of wave radiation stresses, wind and tide;

(ii) lateral mixing is incorporated through conventional 2-D modelling lateral shear and (eddy) viscosity representation; and

(iii) flow momentum and continuity are implicitly accounted for to provide longshore consistency in current patterns used in the sand transport computations.

Accordingly, the modelling software used in this project incorporated all of these features. It was necessary to have the capability to incorporate the 2-D wave propagation (refraction/diffraction/breaking) software into the hydrodynamics and sand transport software. A special purpose system was therefore developed for that purpose, illustrated diagrammatically in Figure 4.1.

Key elements in the software system are:

TWOPRO:	A pre and post processor for preparing the models, calculating wave radiation stresses and sand transport, and exporting results plot files.
RCPR:	A utilities package which facilitates linkage between TWOPRO and the wave propagation software RCPWAVE. It also allows nesting and rotation of RCPWAVE sub-grids to overcome the implicit limitations of the software.
TUFLOW:	The 2-Dimensional hydrodynamics software used to calculate currents in the nearshore coastal zone, with dynamic linkage to the 1-Dimensional hydrodynamics software ESTRY.
ESTRY:	The 1-Dimensional hydrodynamics software used to represent the entire river estuary, linked dynamically to TUFLOW.

This software system provides 2-Dimensional (in plan) information on sand transport for each wave/wind scenario modelled. For this project, such modelling was undertaken for a wide range of potentially occurring scenarios differentiated in terms of:

- deep water wave height;
- deep water wave period;
- deep water wave direction;
- associated wind; and
- East Australian Current

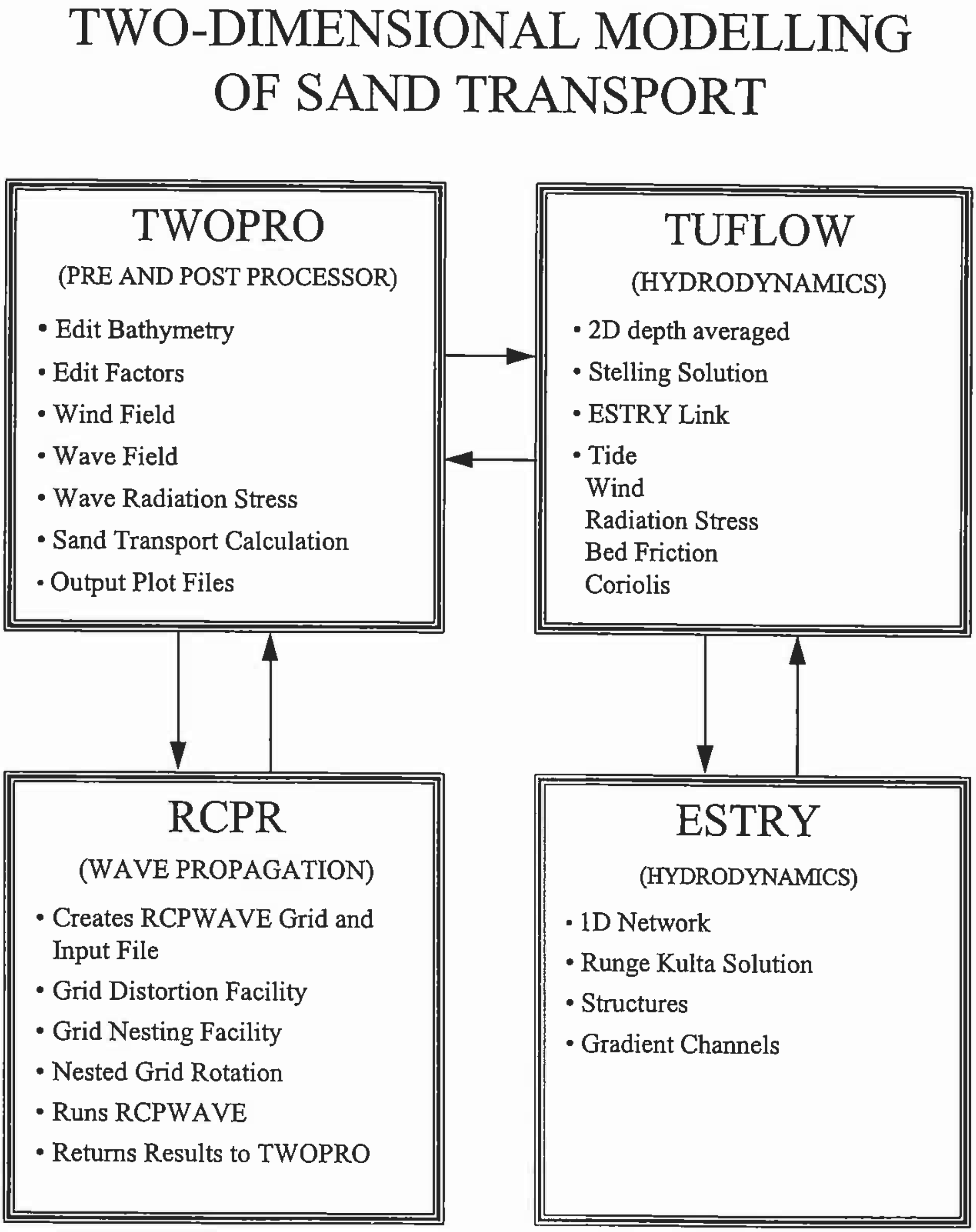


Figure 4.1 Longshore Transport Modelling Software

4.4 Two Dimensional Model Establishment

The regional extent of the two dimensional model was designed primarily to facilitate determination of local wave/current induced sand transport at the designated calculation sites without excessive influences induced at the boundaries. It is practicable and acceptable to restrict the spatial extent of the model in this case where relatively local wave radiation stress forces dominate the local current generation.

Thus, the primary model boundary was nested within that used for wave propagation analyses, extending from Letitia Spit just north of Fingal to Currumbin, and offshore to beyond the 50 metre depth contour. This model extended 6.5 and 4.5 kilometres in the east-west and north-south directions respectively, with a grid resolution of 50 metres as illustrated in Figure 4.2. Nested sub-models for particular regions were also established with 25 metre grid resolution for use in low to moderate sea and swell wave case analyses where local wave breaking dominates the coastal processes.

The model boundaries were set as tide levels which could be either held constant or varied sinusoidally depending on the particular test case requirement. The tide was assumed constant along the eastern boundary, with a lag in the east-west direction to simulate the shoreward propagation of the tidal wave. Where required, the two dimensional model was linked dynamically to the one dimensional model of the Tweed River to represent the flow across the entrance bar area.

Nearshore boundary water level clamping was accepted in this case. Thus the local effects of wave set-up at the immediately adjacent beach areas caused unrealistic local boundary current circulations there. Such effects did not propagate significantly into the model area and did not affect the required calculation results.

The influence of the East Australian Current was modelled by 'tilting' the model as appropriate to give a north-south current through the region at velocities corresponding to those measured in various locations in the Point Danger area.

4.5 Two-Dimensional Longshore Transport Modelling Results

Examples of the results of the 2-Dimensional modelling of longshore transport are presented herein to illustrate key factors to be considered both in assessing the environmental impacts of the bypassing project and in determining performance criteria. Unfortunately, little data is available for any realistic validation of the model results. Only the end product longshore transport rates, estimated over many years to average about 500,000m^3/year through this beach system may be used as an ultimate check on the quantitative results of this work.

Nevertheless, as a minimum, the modelling does provide useful insights into the qualitative processes taking place and, because it has been comprehensively undertaken, is most likely quite reliable quantitatively as well. Discussions of the key results follows.

4.5.1 Beach System Currents

The methodology by which the wave-induced longshore currents in and near the surfzone are generated is different and more theoretically fundamental and comprehensive than that usually adopted for computation of longshore sand transport rates. Commonly, a generalised formulation for longshore current on a plane sloping beach is used, together with an assumed lateral mixing coefficient which yields an acceptable standardised cross-shore distribution of the longshore current. For example, the relationship of Longuet-

Higgins (1972) is commonly used, with a lateral mixing coefficients in the range of 0.2 to 0.5, yielding a cross-shore velocity distribution as shown in Figure 4.3(a).

In contrast, the present modelling procedure uses the fundamental two-dimensional radiation stress values derived from the modelled wave propagation and breaking patterns as forcing for the wave-induced currents. Lateral mixing is implicitly achieved via the two-dimensional hydrodynamic modelling process incorporating appropriate eddy viscosity for sea water.

Comparison of a typical model result with the results obtained from the formulations of Longuet-Higgins (1972) and Battjes (1974) for a straight beach with plane sloping nearshore profile is shown in Figure 4.3(a). An equivalent result for a barred profile is shown in Figure 4.3(b). These results confirm that the modelling procedure provides a good representation of the nearshore current pattern in comparison with other methods, and highlights the advantages of the procedure for application in complex bathymetry areas.

Typical patterns of wave/wind induced nearshore currents along the beach system as derived from the modelling are presented in Figures 4.4(a) and 4.4(b). These show velocity vectors for typical southeast swell (2m) with mean spring ebb and flood tide flow from the Tweed River with the pre-dredging entrance bar bathymetry, and northeast sea (2m) conditions as well as east storm (4m) conditions respectively.

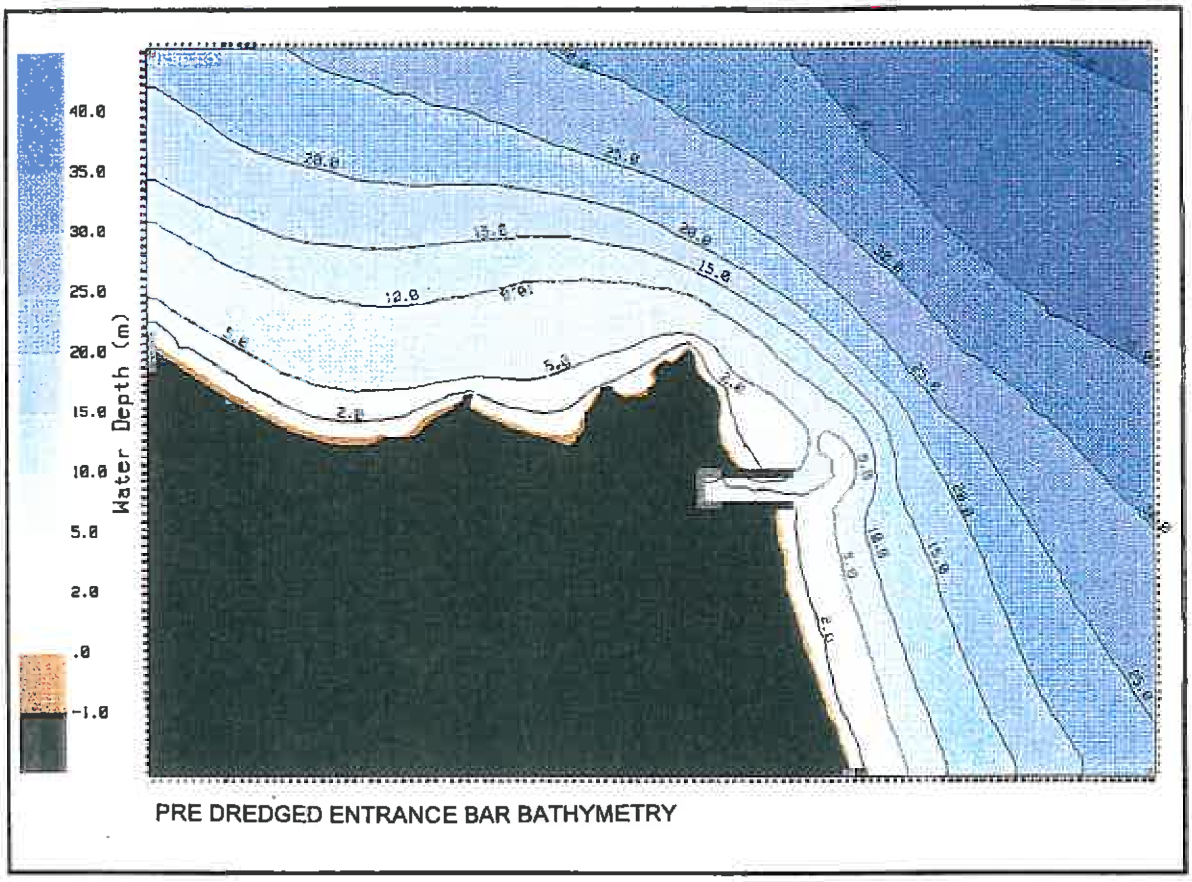
Key features of these results are summarised as follows:

- Southeast swell generates wave-induced longshore currents predominantly along the ocean beaches south of Point Danger and past the headlands at Snapper Rocks, Greenmount and Kirra. The north-facing southern Gold Coast beaches generally have lower current speeds where the waves break more shore-normal.
- Northeast waves generate divergent current at Point Danger, downcoast along Duranbah and Letitia Spit and upcoast along the Gold Coast beaches.
- Storm related currents, driven by both wave and wind forces, are stronger and extend further into deeper water than for the swell. The more southerly storm winds tend to force the current northward past Point Danger, tending to separate from the wave-induced nearshore current at Rainbow Bay.
- East-northeast storm conditions indicate generation of an upcoast current along the river mouth bar off Duranbah, with a downcoast current nearshore at Duranbah and along Letitia Spit. This upcoast current in the offshore area appears to be enhanced by the ebb jet flow from the river and the input of flow to sustain the strong upcoast current (up to 2m/s) along Snapper Rocks and Rainbow Bay.
- The strongest longshore currents along the southern Gold Coast beaches result primarily from the higher east to northeast wave and wind conditions.

4.5.2 Longshore Sand Transport

Longshore sand transport rates have been calculated at seven (7) locations within the study area as shown in Figure 4.5. These have been derived from the 2-Dimensional modelling results. The sand transport formulation utilises the assessed wave field in combination with the modelled currents to compute sand transport rates and directions. Thus sand transport is calculated to be in the direction of the current at any specific location.

The tidal influence on nearshore currents has been ignored for the general longshore transport calculations at each of the seven designated beach locations. Clearly this could not be presumed at the river entrance, and may also influence the results for Duranbah.



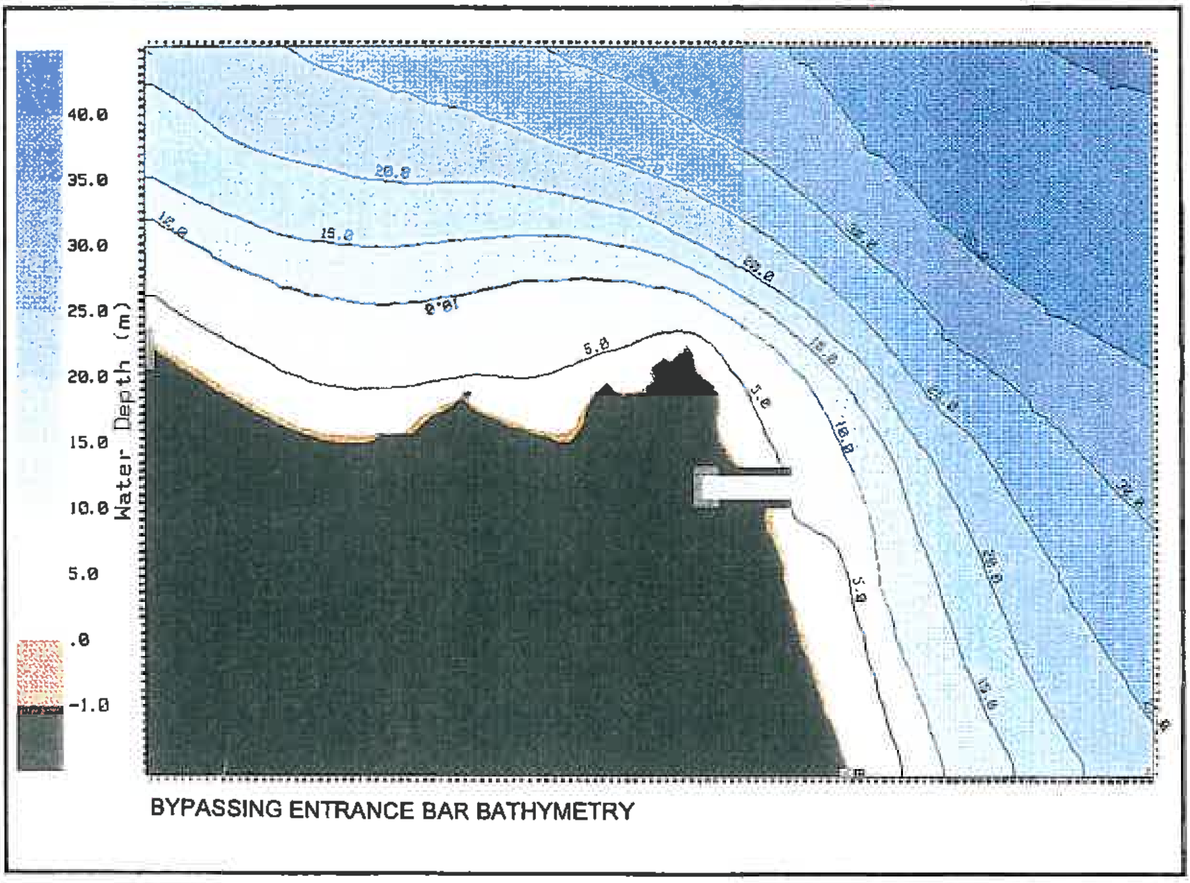


Figure 4.2
Two-Dimensional Sand Transport Model Extent

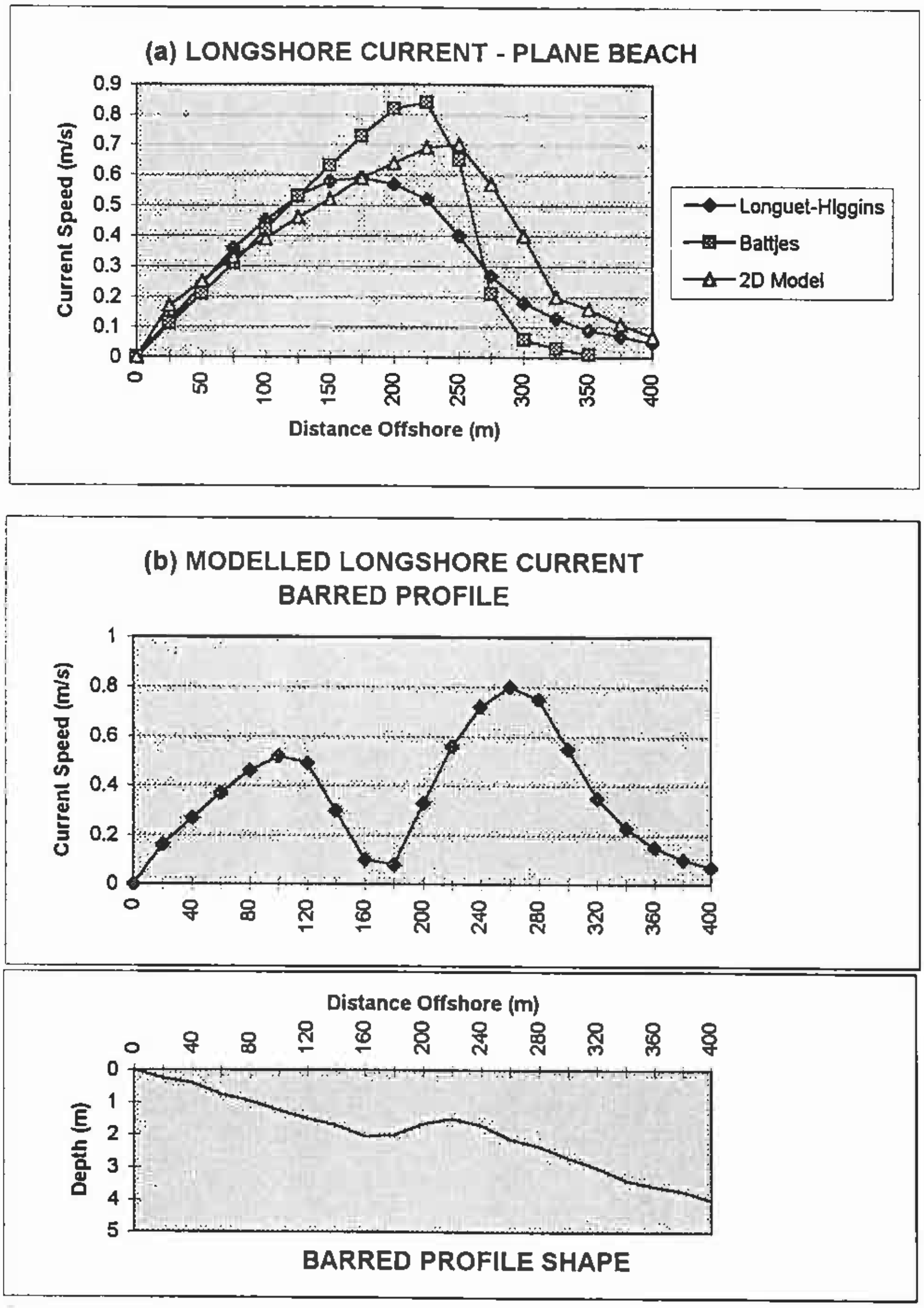
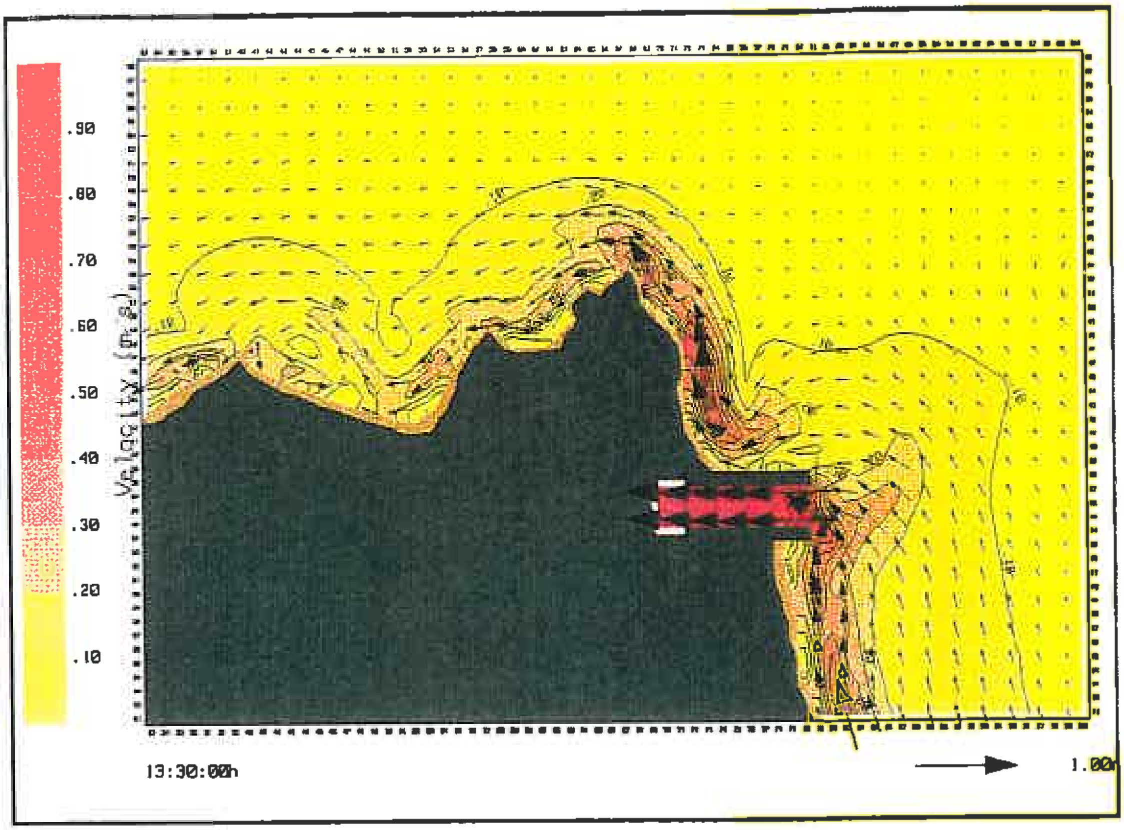
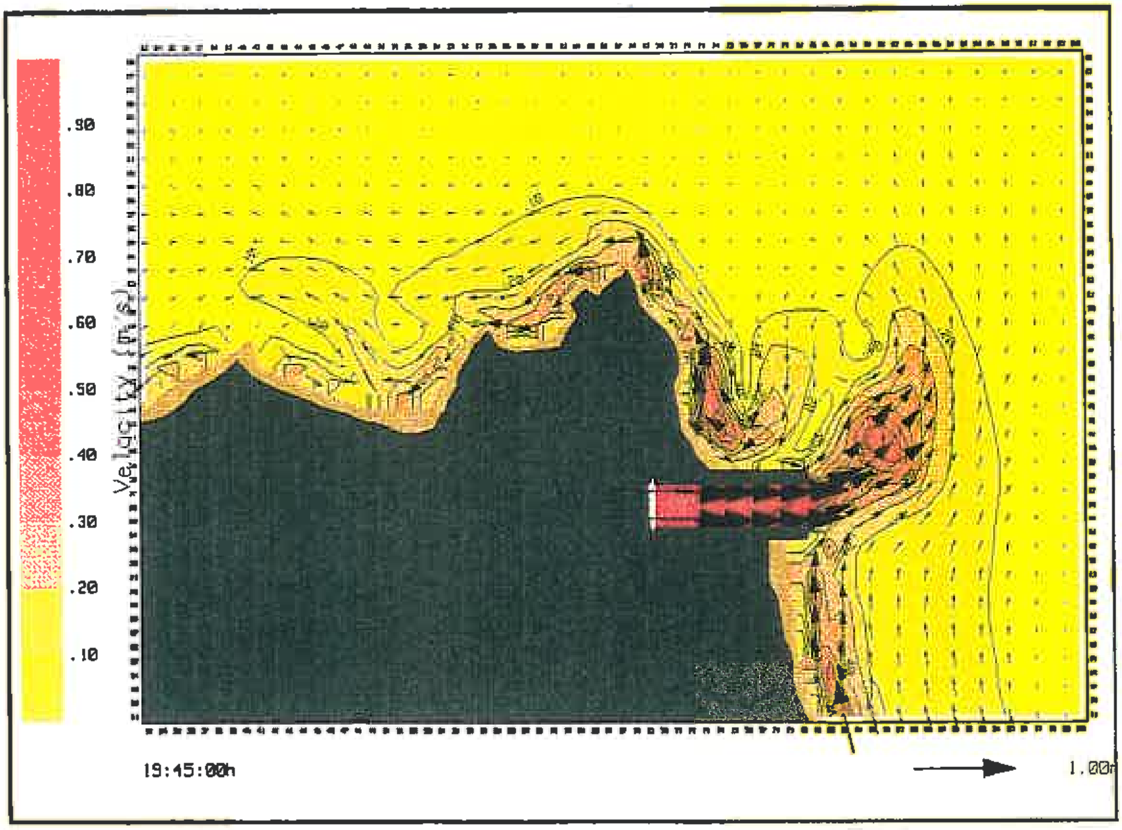


Figure 4.3
Modelled Wave-Induced Longshore Current Distribution
(Ho = 2m; T = 7.0s; θo = 30°)

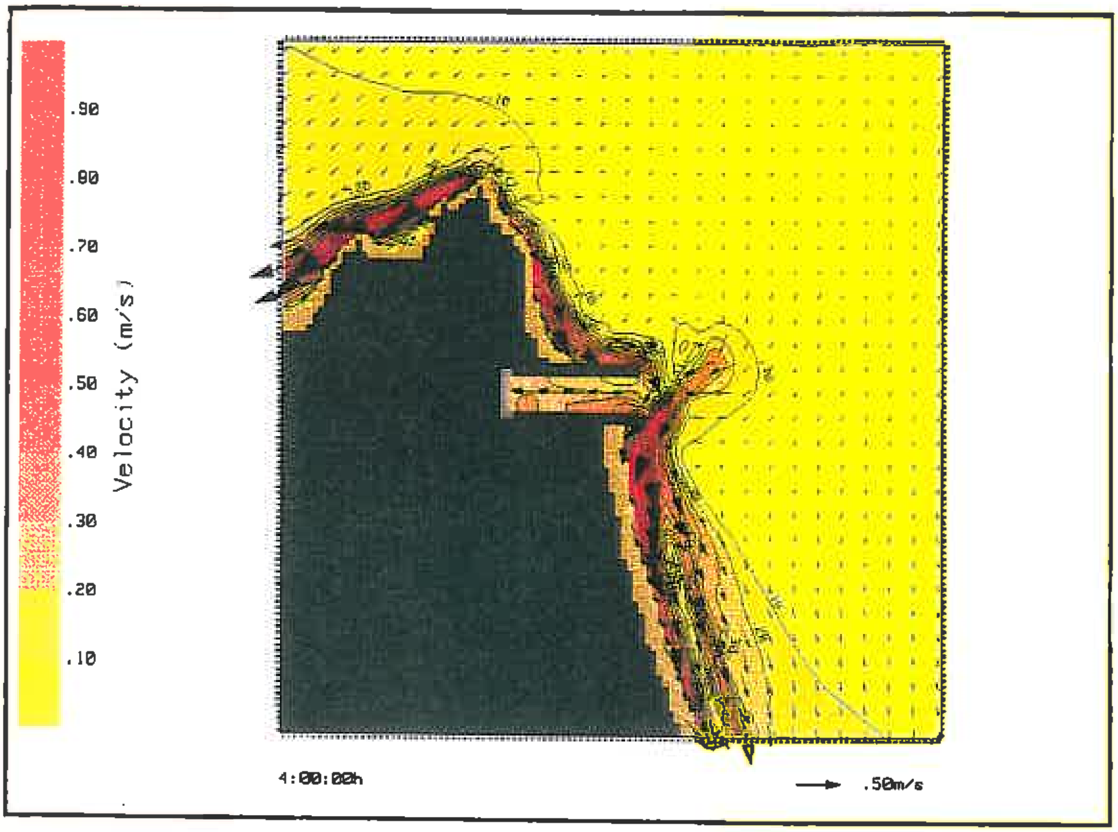


FLOOD TIDE

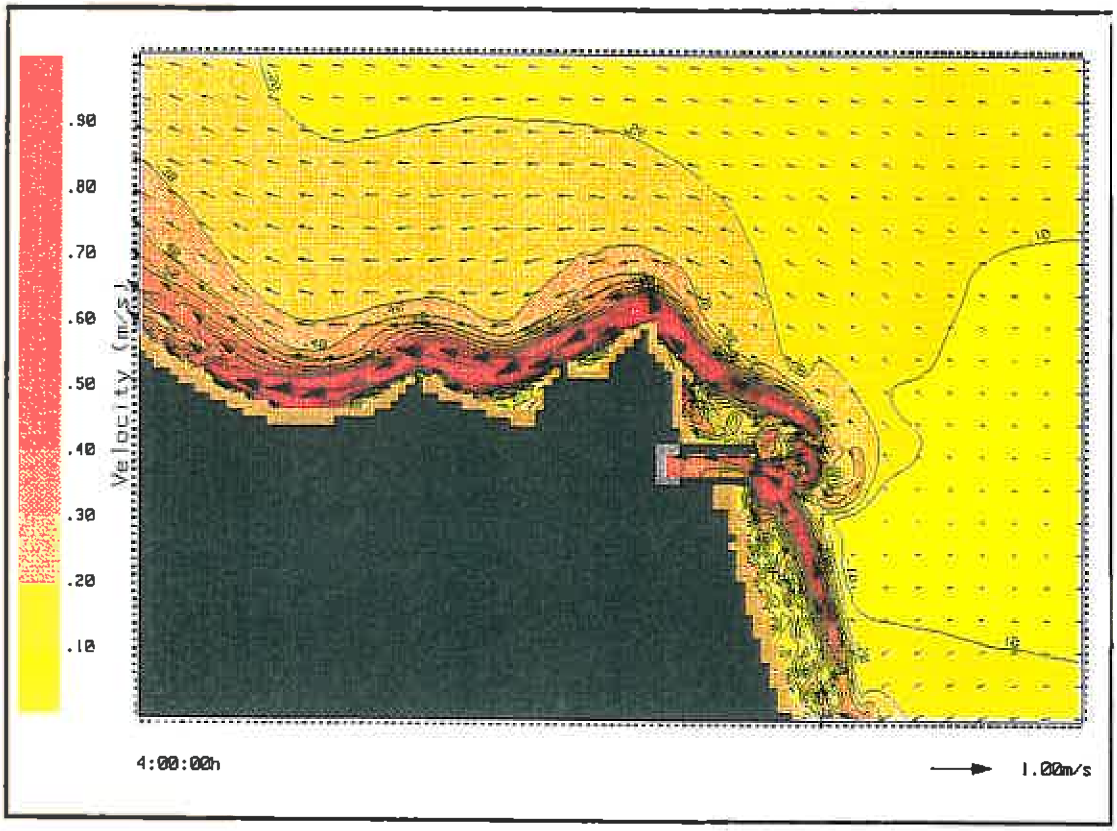


EBB TIDE

Figure 4.4(a)
Modelled Nearshore Currents - Pre-Dredged Bathymetry SE Swell Waves - With River Tidal Currents



NORTHEAST SEA WAVES



EAST STORM (4m) WAVES

Figure 4.4(b)
Modelled Nearshore Currents - Pre-Dredged Bathymetry
NE Sea & E Storm Waves - No River Current

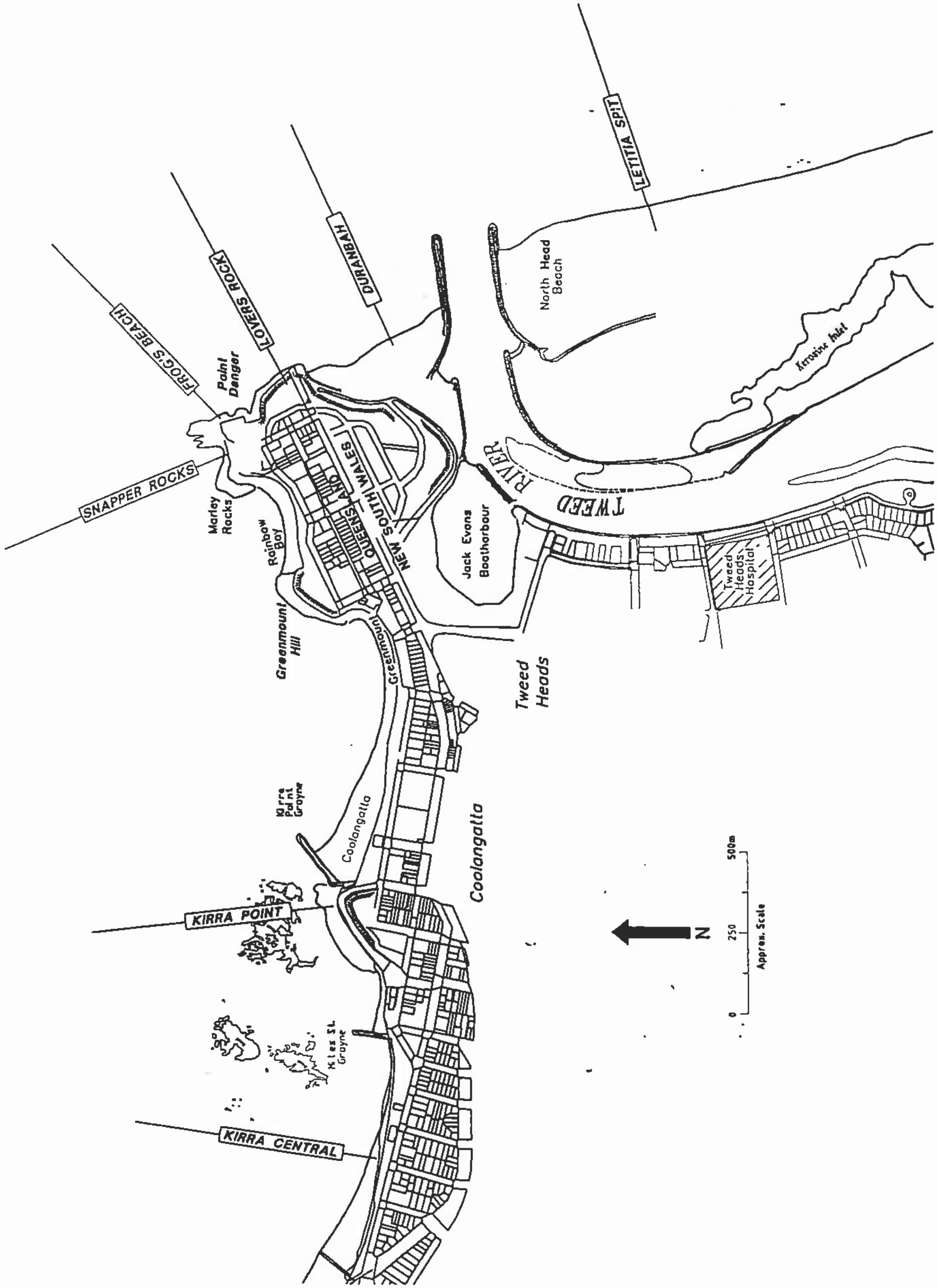


Figure 4.5
Longshore Transport Calculation Locations

The modelled two-dimensional sand transport patterns associated with southeast swell, northeast sea and typical storm conditions are presented in Figures 4.6 and 4.7. It can be seen that sand transport is generally concentrated predominantly within the surfzone and immediate nearshore areas. Significant transport (more than 0.05m^3/m/hour) extends out to about 5 meters water depth for 2m swell and to about 15 metres depth for 4m storm events. Clearly for smaller waves, sand transport would occur along a quite narrow nearshore zone, while for extreme cyclone events it would extend well offshore.

4.6 Calculation of Sand Transport Time Series

A unique feature of the modelling undertaken for this project is generation of a time series of longshore sand transport from the input time series of prevailing waves. This required development of special-purpose software with the facility to:

(i) interpolate and extrapolate sand transport rates for each input wave condition from the sand transport rates determined for each of the specific wave cases modelled.

(ii) differentiate 'sea' (with wind) and 'swell'.

(iii) incorporate the influence of the EAC with probabilistic occurrence and strength

(iv) allow subdivision of the longshore transport regime into bands to identify the cross-shore variability.

(v) calculate temporal variability of longshore transport in terms of daily, weekly, monthly and annual net and gross rates and probabilities of occurrence.

Input to this software has the form of both the wave time series, expressed in terms of 6-hourly sea and swell (height/period/direction) and array values of sand transport rates corresponding to the specific modelled wave cases. Sand transport rates corresponding to one swell wave/current EAC scenario is also input, from which rates for other swell heights and current speeds are derived.

Input data files containing transport rates derived from the two-dimensional modelling have been established for each of the longshore transport calculation locations. This has been done in terms of transports both for the whole nearshore profile and for specific bandwidths within the profile extent defining designated depth zones. Thus, time series sand transport output is obtained within each of those depth zones as well as for the whole profile.

The wave time series in 6 hourly intervals provides output information also each 6 hours. This is then used to generate results at other specified time intervals appropriate for sand bypassing system design and management. Net and gross transport rates are thus obtained as daily, weekly, monthly and annual time series values. Statistical transport rate occurrence probabilities are then calculated directly from those values.

4.7 Temporal and Spatial Variability of Longshore Transport

The results of the longshore transport modelling are presented in time series form in Appendix B. Tabulated transport results are presented as follows:

- Table 4.1 - Annual Net and Gross Sand Transport (Total each year and average)
- Table 4.2 - Annual Net Sand Transport (Distribution average)
- Table 4.3 - Annual Gross Sand Transport (Distribution average)
- Table 4.4 - Daily Longshore Transport Probabilities

- Table 4.5 - Weekly Longshore Transport Probabilities
- Table 4.6 - Monthly Longshore Transport Probabilities

The results in Tables 4.2 and 4.3 are presented in terms of both whole profile gross/net results and incremental transport components in the cross-shore depth zones:

0-2m below MSL
2-4m below MSL
4-8m below MSL
8-12m below MSL
>12m below MSL

These results describe the cross-shore variability of the longshore transport and the variability from year to year.

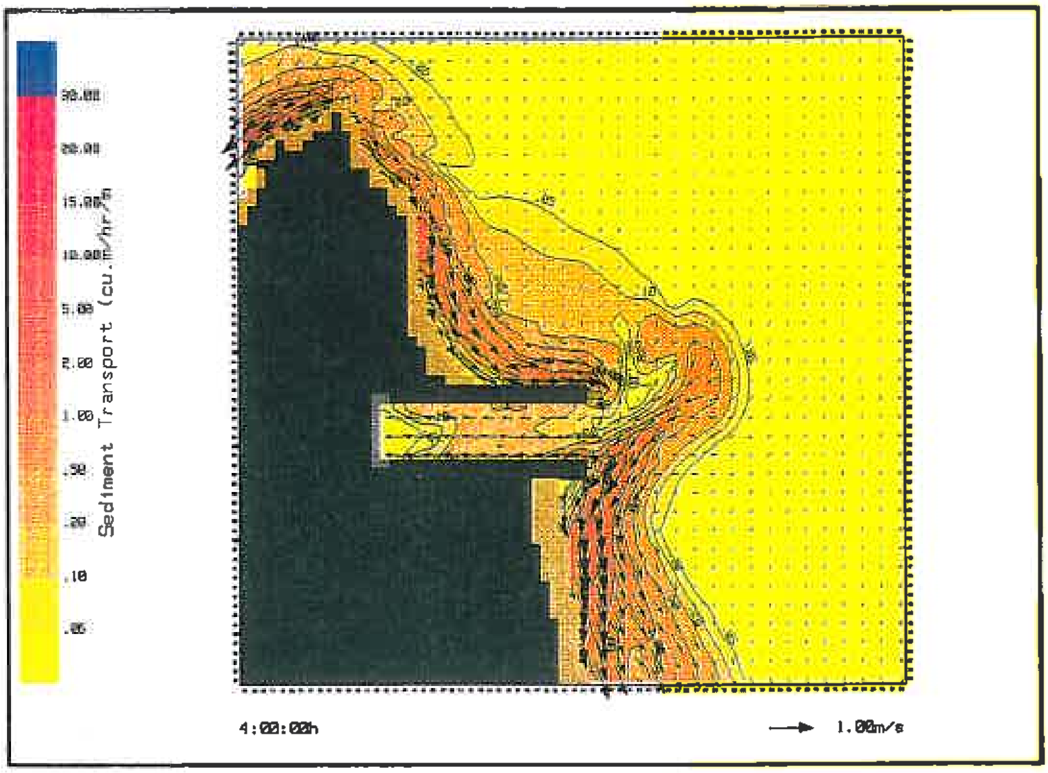
The longshore transport occurrence probabilities are also plotted in Appendix C.

Probabilities of exceedance of weekly and monthly net longshore transports are presented graphically in Figure 4.8(a) and (b) respectively. These plots illustrate the rather different relative influences of low, moderate and high transport periods to the overall average net transport potential at each site. The results for Duranbah (not plotted for clarity) conforms very closely with that for Letitia Spit in each case, despite the significant difference in cross-shore distribution of the longshore transport at those sites, as shown in Table 4.2.

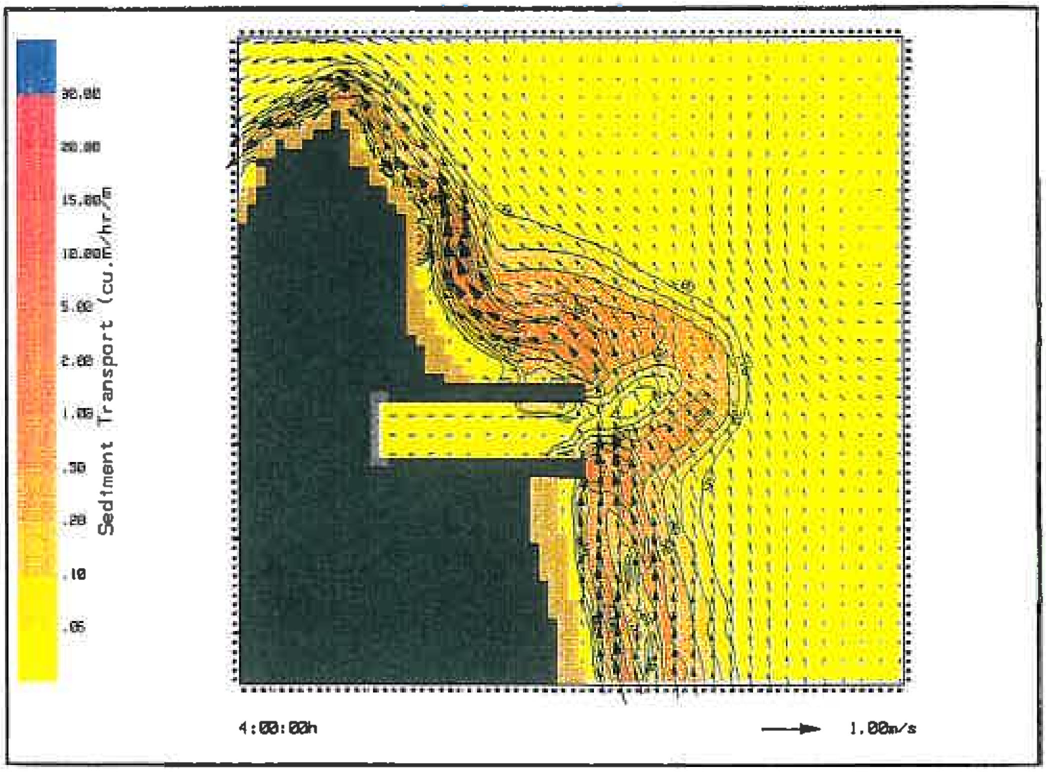
Seasonal patterns of longshore transport at the Letitia Spit and Snapper Rocks sites are illustrated as monthly mean net and gross transport rates (Figure 4.9) and individual monthly rates (Figure 4.10). These show a clear pattern of high transport during the summer months, with occasion high transport also in May/June, and typically quite low transport from August through to November.

Table 4.1 Annual Net and Gross Sand Transport (m^3/year)

NORTH KIRRA:	NET	UPCOAST	DOWNCOAST
1990	460000	460000	0
1991	279000	279000	0
1992	458000	458000	0
1993	478000	478000	0
1994	419000	419000	0
1995	510000	510000	0
AVERAGE	**434000**	**434000**	**0**
KIRRA	**NET**	**UPCOAST**	**DOWNCOAST**
1990	505000	505000	0
1991	337000	337000	0
1992	487000	487000	0
1993	527000	527000	0
1994	514000	514000	0
1995	590000	590000	0
AVERAGE	**493000**	**493000**	**0**

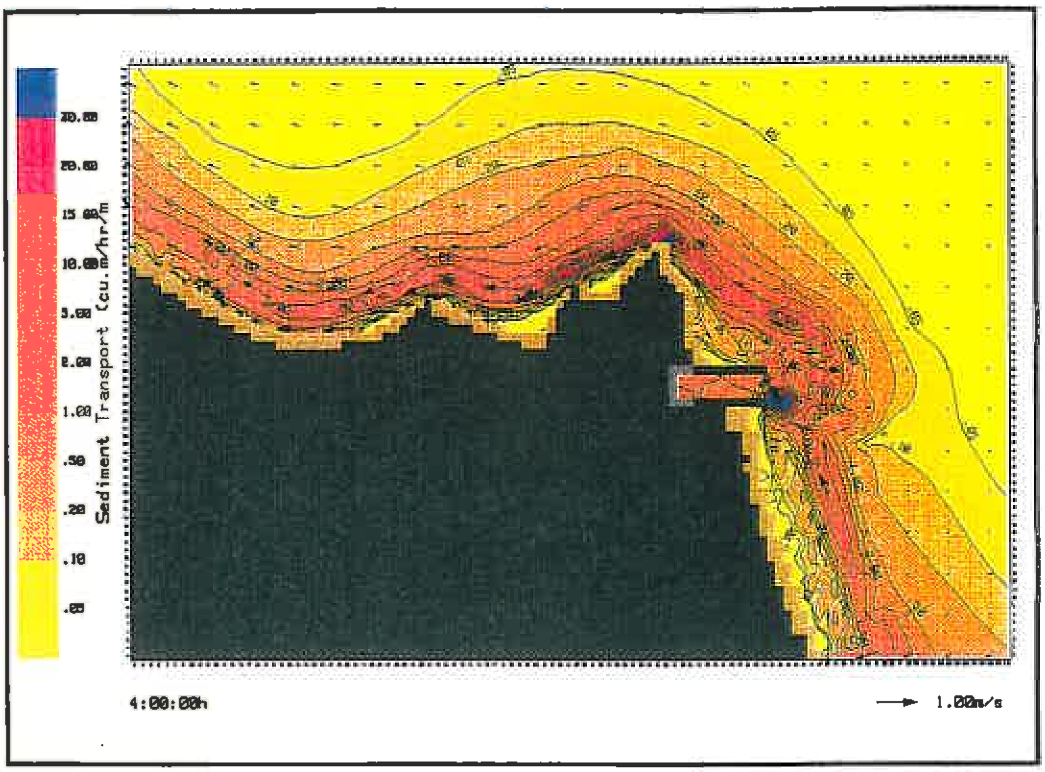


NORTH EAST SEA (2m6s) - EXISTING

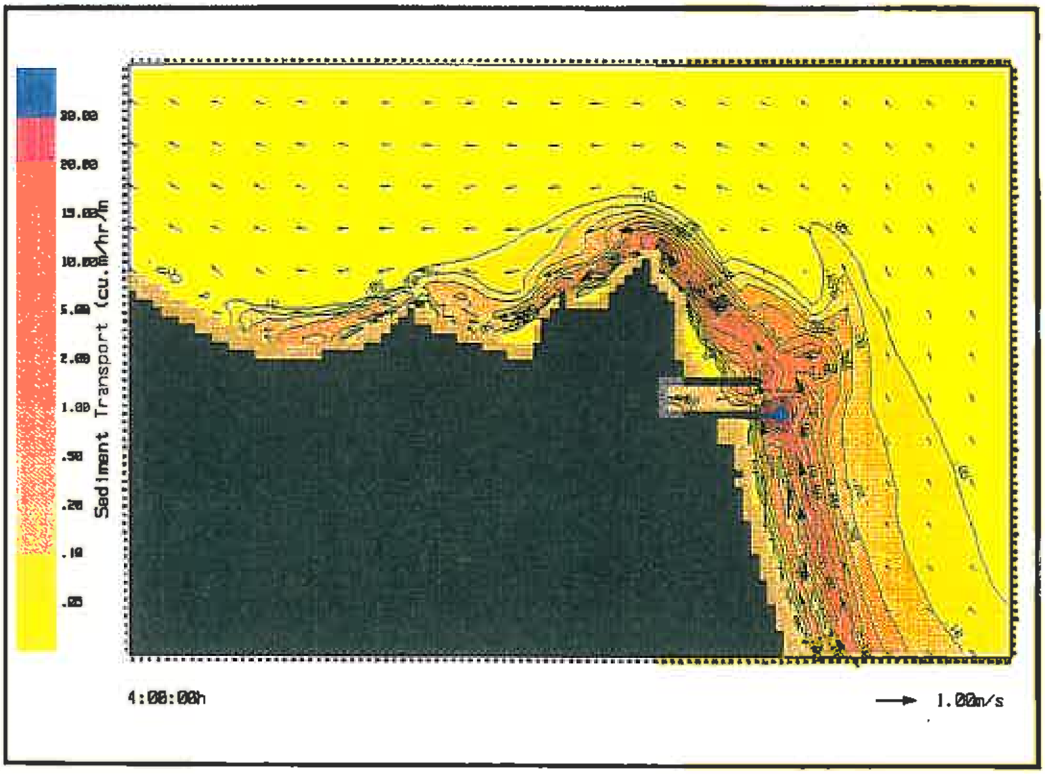


SOUTH EAST SWELL (2m10s) - EXISTING

Figure 4.6
Modelled Sand Transport Pattern
- Southeast Swell and Northeast Sea



EAST STORM (4m9s) - EXISTING



SOUTH EAST STORM (4m9s) - EXISTING

Figure 4.7
Modelled Sand Transport Pattern - Storm Conditions

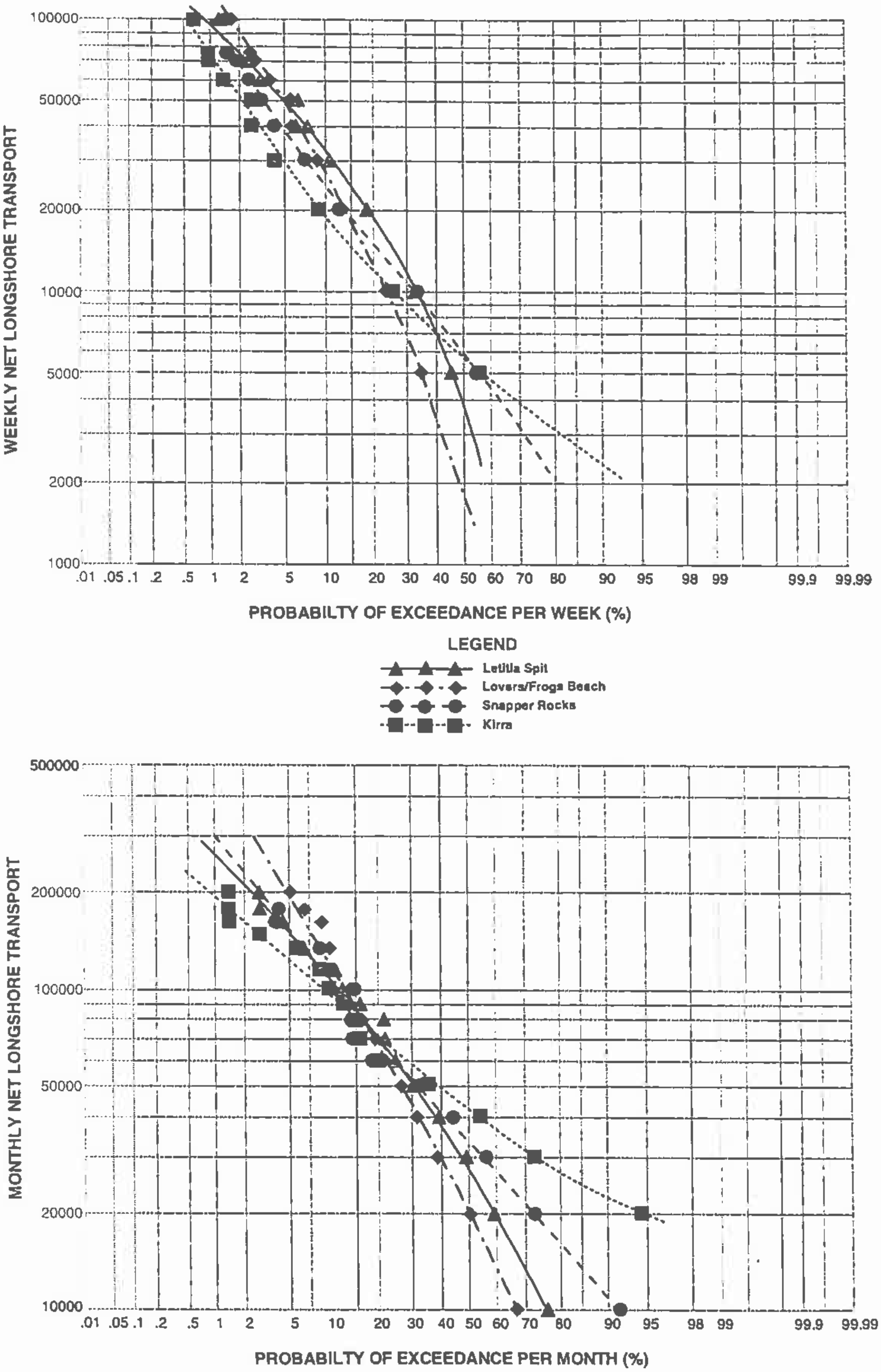
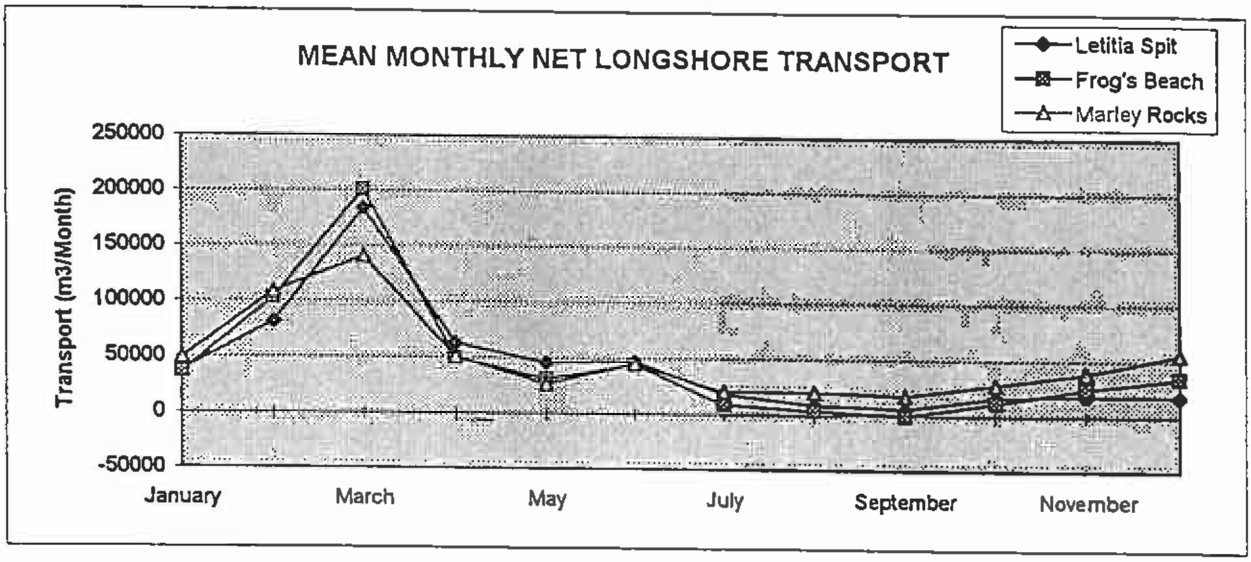


Figure 4.8
Probabilities of Exceedance - Weekly and Monthly Net Longshore Transport



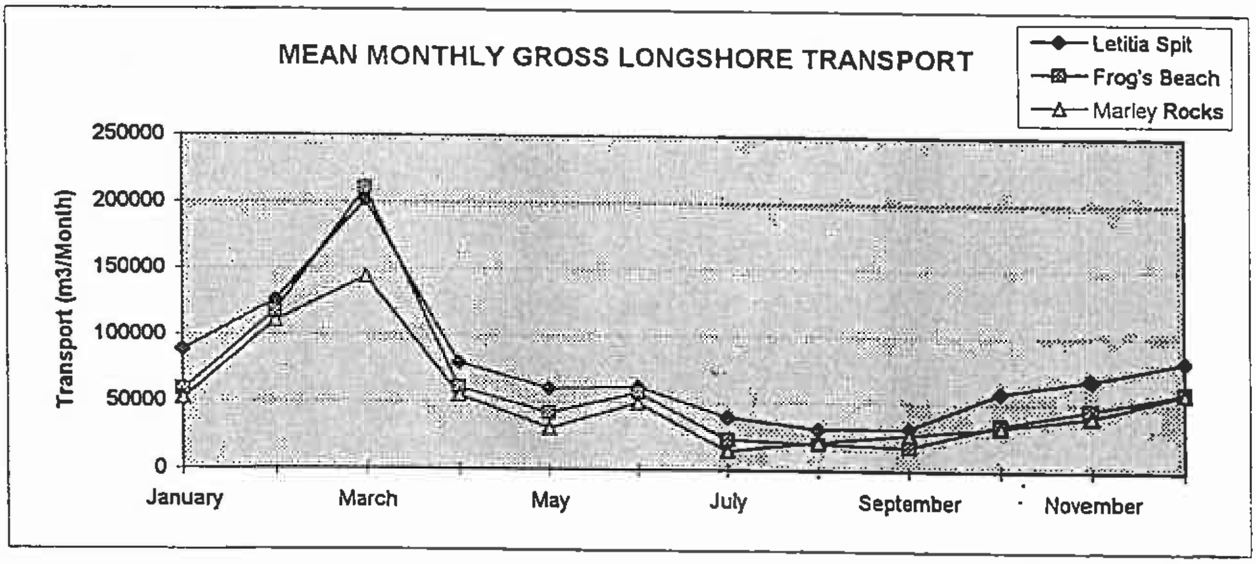
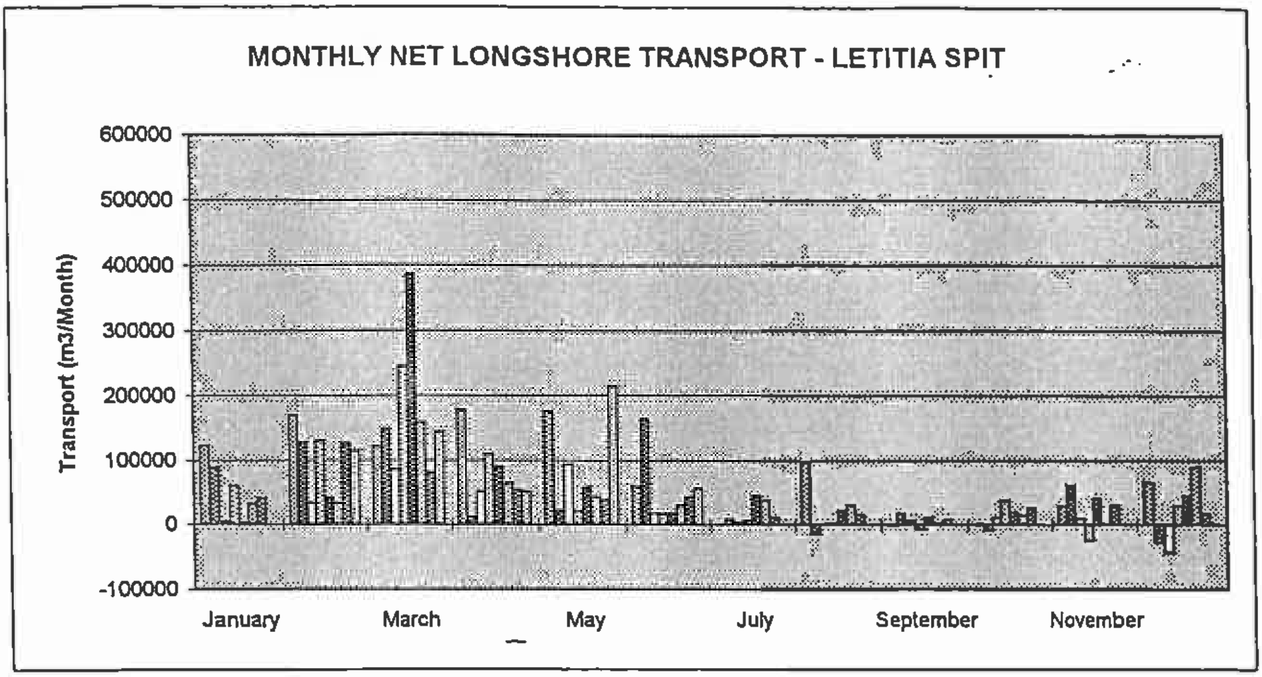


Figure 4.9
Mean Monthly Longshore Transport



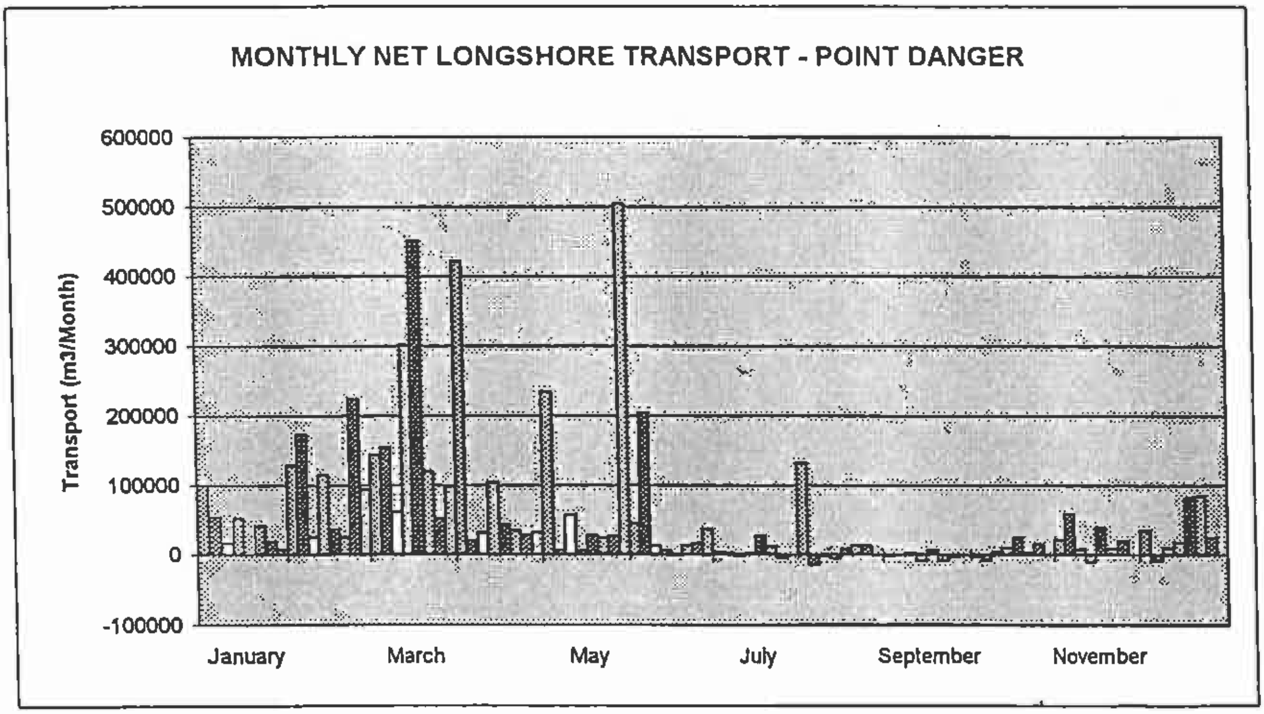


Figure 4.10
Monthly Longshore Transport Rates For the Years 1989 to 1996

Table 4.1 Annual Net and Gross Sand Transport (m³/year) (Cont)

SNAPPER ROCKS:	NET	UPCOAST	DOWNCOAST
1990	677000	696000	-19000
1991	367000	381000	-14000
1992	635000	647000	-12000
1993	703000	711000	-8000
1994	594000	604000	-10000
1995	630000	644000	-14000
AVERAGE	**601000**	**614000**	**-13000**
FROGS BEACH	**NET**	**UPCOAST**	**DOWNCOAST**
1990	638000	724000	-86000
1991	204000	281000	-77000
1992	583000	671000	-88000
1993	750000	822000	-72000
1994	365000	444000	-79000
1995	430000	505000	-75000
AVERAGE	**495000**	**575000**	**-80000**
LOVERS ROCKS:	**NET**	**UPCOAST**	**DOWNCOAST**
1990	707000	789000	-82000
1991	305000	378000	-73000
1992	680000	764000	-84000
1993	889000	962000	-73000
1994	502000	579000	-77000
1995	519000	586000	-67000
AVERAGE	**600000**	**676000**	**-76000**
DURANBAH:	**NET**	**UPCOAST**	**DOWNCOAST**
1990	653000	798000	-145000
1991	258000	427000	-169000
1992	640000	789000	-149000
1993	773000	922000	-149000
1994	569000	718000	-149000
1995	565000	680000	-115000
AVERAGE	**576000**	**722000**	**-146000**
LETITIA SPIT:	NET	UPCOAST	DOWNCOAST
1990	592000	745000	-153000
1991	267000	451000	-183000
1992	613000	769000	-156000
1993	783000	933000	-150000
1994	521000	670000	-149000
1995	494000	622000	-128000
TOTAL	545000	698000	-153000

Table 4.2 Annual Net Longshore Transport Years 1990 - 1995

Net Transports						
	0 - 2m	**2 - 4m**	**4 - 8m**	**8 - 12m**	**>12m**	**TOTAL**
Kirra	68,380 (14%)	252,240 (51%)	136,370 (28%)	27,860 (6%)	8,140 (2%)	493,000
Snapper Rocks	188,600 (31%)	268,260 (45%)	132,000 (22%)	10,470 (2%)	1,230 (0.2%)	601,000
Frog's Beach	16,790 (3%)	316,170 (64%)	186,850 (38%)	-16,360 (-3%)	-8,440 (2%)	495,000
Lovers Rocks	72,380 (12%)	370,770 (62%)	181,050 (30%)	-11,090 (-2%)	-13,100 (-2%)	600,000
Duranbah	-11,830 (-2%)	357,950 (62%)	236,390 (41%)	8,570 (2%)	-15,090 (-3%)	576,000
Letitia Spit	145,720 (27%)	308,930 (57%)	114,260 (21%)	2,040 (0.4%)	-24,950 (-5%)	545,000

Table 4.3 Annual Gross Longshore Transport Years 1990-1995

Gross Transports						
	0 - 2m	2 - 4m	4 - 8m	8 - 12m	>12m	TOTAL
Kirra	68,380 (14%)	252,240 (51%)	136,370 (28%)	27,860 (6%)	8,140 (2%)	493,000
Snapper Rocks	188,600 (29%)	269,500 (42%)	135,200 (5%)	31,500 (5%)	22,200 (3%)	647,000
Frog's Beach	44,735 (7%)	317,990 (49%)	211,790 (32%)	36,310 (6%)	44,170 (7%)	655,000
Lovers Rocks	96,030 (13%)	382,970 (51%)	194,100 (26%)	38,630 (5%)	40,270 (5%)	752,000
Duranbah	161,950 (19%)	377,910 (44%)	238,120 (27%)	48,080 (6%)	41,950 (5%)	868,000
Letitia Spit	213,580 (25%)	403,650 (47%)	154,400 (18%)	30,660 (4%)	48,690 (6%)	851,000

Table 4.4 Daily Net Longshore Transport Probabilities (%)

Transport (m^3/day)	Letitia Spit	Duranbah	Lovers Rock/ Frog's Beach	Snapper Rocks	Kirra
<-10,000	0.15	0.10	-	-	-
-10,000 to -8,000	0.09	0.23	-	-	-
-8,000 to -6,000	0.27	0.28	0.05	-	-
-6,000 to -4,000	0.91	0.82	0.07	-	-
-4,000 to -2,000	4.15	3.65	0.90	-	-
-2,000 to - zero	38.75	36.65	49.03	19.26	-
zero to 2,000	33.96	34.92	34.57	61.30	84.61
2,000 to 4,000	8.62	9.45	6.12	11.14	9.86
4,000 to 6,000	3.88	4.92	2.83	3.38	2.24
6,000 to 8,000	2.92	2.88	1.81	1.32	1.10
8,000 to 10,000	1.60	1.78	0.98	1.14	0.41
10,000 to 12,000	1.32	1.10	0.69	0.36	0.60
12,000 to 14,000	0.87	0.50	0.44	0.41	0.09
14,000 to 16,000	0.41	0.64	0.40	0.09	0.37
16,000 to 18,000	0.36	0.14	0.21	0.18	0.10
18,000 to 20,000	0.27	0.41	0.16	0.14	0.05
20,000 to 22,000	0.19	0.32	0.15	0.09	0.19
22,000 to 24,000	0.23	0.27	0.18	0.09	0.09
>24,000	1.05	0.96	1.50	1.00	0.32

Table 4.5 Weekly Longshore Transport Probabilities (%)

Net Transport (m^3/week)	Letitia Spit	Duranbah	Lovers Rock/ Frog's Beach	Snapper Rocks	Kirra
<-30,000	0.32	0.32	-	-	-
-30,000 to -25,000	0.32	0.00	-	-	-
-25,000 to -20,000	0.00	0.64	-	-	-
-20,000 to -15,000	1.28	0.64	-	-	-
-15,000 to -10,000	0.64	0.96	-	-	-
-10,000 to -5,000	7.99	7.03	2.88	-	-
-5,000 to zero	23.64	22.04	36.43	5.11	-
zero to 5,000	21.09	22.04	24.72	40.26	46.33
5,000 to 10,000	14.38	13.74	12.14	22.68	28.43
10,000 to 15,000	6.07	7.99	5.59	13.74	10.86
15,000 to 20,000	6.07	5.75	4.15	5.43	5.43
20,000 to 25,000	6.07	5.43	2.40	3.51	3.19
25,000 to 30,000	1.92	2.56	1.92	1.92	1.60
30,000 to 35,000	1.60	1.92	1.28	1.92	1.60
35,000 to 40,000	1.60	1.60	0.96	1.28	0.00
40,000 to 45,000	0.96	0.96	0.64	0.96	0.00
45,000 to 50,000	0.32	1.92	0.48	0.00	0.00
50,000 to 55,000	1.60	0.32	1.28	0.32	0.32
55,000 to 60,000	1.28	0.32	0.48	0.64	0.96
60,000 to 65,000	0.32	0.64	0.48	0.00	0.00
65,000 to 70,000	0.00	0.32	0.32	0.32	0.32
70,000 to 75,000	0.96	0.64	0.16	0.32	0.00
>75,000 to 100,000	0.32	1.28	0.80	0.00	0.32
>100,000	1.28	0.96	1.76	1.60	0.64

Table 4.6 Monthly Net Longshore Transport Probabilities (%)

Net Transport (m^3/week)	Letitia Spit	Duranbah	Lovers Rock/ Frog's Beach	Snapper Rocks	Kirra
-50,000 to -40,000	1.39	1.39	-	-	-
-40,000 to -30,000	0.00	0.00	-	-	-
-30,000 to -20,000	2.78	2.78	-	-	-
-20,000 to -10,000	1.39	2.78	4.17	-	-
-10,000 to zero	5.56	5.56	13.20	-	-
zero to 10,000	15.28	12.50	16.67	9.72	5.56
10,000 to 20,000	15.28	13.89	15.98	19.44	23.61
20,000 to 30,000	9.72	8.33	11.81	15.28	18.06
30,000 to 40,000	9.72	12.50	6.95	12.50	18.06
40,000 to 50,000	9.72	11.11	5.56	12.50	15.28
50,000 to 60,000	5.56	5.56	4.86	12.50	4.17
60,000 to 70,000	2.78	1.39	2.09	4.17	1.39
70,000 to 80,000	0.00	5.56	3.47	0.00	1.39
80,000 to 90,000	5.56	4.17	2.09	0.00	2.78
90,000 to 100,000	2.78	0.00	0.00	0.00	1.39

Net Transport (m^3/week)	Letitia Spit	Duranbah	Lovers Rock/ Frog's Beach	Snapper Rocks	Kirra
100,000 to 120,000	1.39	1.39	3.48	4.17	2.78
120,000 to 140,000	4.17	1.39	0.00	1.39	2.78
140,000 to 160,000	2.78	1.39	2.09	4.17	1.39
160,000 to 180,000	1.39	1.39	2.09	0.00	0.00
180,000 to 200,000	0.00	4.17	0.70	0.00	0.00
>200,000	2.78	2.78	4.87	4.17	1.39

Review of the above results indicate the following key conclusions:

(i) The calculated net longshore transport throughout the study area is about 500,000 to 600,000m^3 per year at all locations, averaging about 550,000m^3/year. On the basis that this is about 13% higher than the long term average, the previously assessed long term average annual net transport rate of 500,00m^3/year is supported.

(ii) There is substantial variability in the annual net and gross longshore transport rates, calculated to be typically in the range 250,000m^3/year (1991) to 800,000m^3/year (1993). The calculated net rate for Letitia Spit in 1989 is about 1 million m^3/year considered probably at the upper limit likely to occur from time to time.

(iii) Longshore transport rates are highly variable over time, with most transport occurring during the higher swell wave and storm wave periods.

(iv) There is significant spatial variability of longshore transport at different locations along the beach unit at any time. This leads to periods of significant variability of beach sand volumes at some beaches, particularly Snapper Rocks where sustained northeast sector waves, in conjunction with EAC effects, may result in sustained sand starvation. This is compensated in times of strong southeast sector waves when surplus sand moves past Point Danger.

(v) The vast majority of longshore transport occurs in water depths less than 4 metres, particularly in depths of 2-4 metres. Nevertheless, significant (typically 20%) of transport occurs in depths of 4-8 metres. Progressively less transport occurs further offshore, and longshore transport at the more exposed beaches is influenced by the EAC.

(vi) Gross longshore transport rates are highest at Letitia Spit, where significant downcoast transport occurs from time to time, and least at Kirra Point where transport is always upcoast (that is, the gross and net transport rates are equal).

These results are transport potentials as calculated for the particular nearshore bathymetries provided for in the two-dimensional model. Thus, they may not properly reflect the actual sand transport rates at locations adjacent to rocky shorelines where the seabed levels varies significantly. Accordingly, the results for the locations at Lovers Rock, Frog's Beach and Snapper (Marley) Rocks should be regarded as indicative only.

Further, while these results appear to indicate a pattern of lower net transport at Kirra and North Kirra, this may not be the case in the prototypes. There are various factors associated with the directional wave climate determination, the wave refraction analysis and the sand transport calculations which would lead to underestimation of the transport in those areas, and overestimation of the transport along the more exposed ocean beaches.

Key factors which contribute to the potential for the present modelling to underestimate the net longshore transport at Kirra, and which would require further data and detailed investigation to assess are as follows:

(i) The adopted BMO model hindcast wave climate, which is considered to underestimate the proportion of swell waves from the easterly sector (refer Section 4.1.6).

(ii) The additional influence (not assessed herein) of the component of wave induced mass (cross-shore) sand transport which is directed along the coast in that area. Preliminary estimates suggest that this component could be (order of) 25,000m^3/year or higher (depending on the complex interaction of wind induced currents and the wave orbital velocities.

(iii) Possible limitations of the present model, particularly for North Kirra, with respect to the proximity of the model boundary, wave refraction to that area and proper representation of the nearshore bar/gutter bathymetry.

It is considered that the present results, together with the previous assessments, are sufficient evidence to conclude that the net transport through the beach system is constant at about 500,000m^3/year.

Alongshore variations in longshore transport cause short to medium term deficits or surpluses of sand in intermediate beach units. These have been assessed in time-series format, and are presented in Figure 4.11 for the sand intake (Letitia Spit) to sand discharge (Frog's Beach) area, and in Figure 4.12 for the area between Frog's Beach and Snapper Rocks.

These results show that the mismatch in the net longshore transport between Letitia Spit and the primarily discharge location is significant but relatively minor. Net storage losses and gains of up to about 100,000m^3 are indicated. The supply from Letitia Spit is more steady, while the transport away is somewhat sporadic, associated with the higher wave events.

However, there are greater differentials in transport between Frog's Beach and Snapper Rocks. These appear to occur during short term high energy storm events which move slugs of sand into the Snapper Rocks area. This then diminishes gradually over a long period as the sand is moved on along the beaches. This data indicates differential 'slug' quantities of up to about 350,000m^3 which are then depleted over a period of some 1 to 3 years.

Figure 4.13 shows the calculated differentials between Snapper Rocks and Kirra. These are relatively minor, the transport at both these sites being strongly dependent on the more east to northeast waves. It should be noted that this does not suggest that the process of sand transport between Snapper Rocks and Kirra occurs as a continuous near uniform rate. The GENESIS modelling as described in Chapter 6 indicates substantial differential pulsing of sand past both Greenmount and Kirra headlands and intermediate storage of sand at Greenmount from time to time.

4.8 Longshore Transport in Storm Events

Storm Wave Conditions

A number of severe storm events in which the significant wave height (H_s) exceeded 5 metres occurred during the period 1989-1996 for which directional wave data has been derived. These include the following:

	Maximum H_s (m)
April 22-29, 1989	6.1
February 1-7, 1990	*5.0
February 25-March 19, 1992	5.1
March 11-25, 1993	7.4
February 12-17, 1995	6.4
February 13-18, 1996	6.2
May 1-7, 1996	6.9

(Note: *indicates that value has been derived from the BMO model results)

Of particular significance in this study are the storms in February and May 1996 which occurred during the river entrance dredging and bathymetric survey monitoring. The available directional wave data for this period are the adopted deep water data combining the recorded Brisbane height/period data with the BMO model hindcast directions, and the nearshore Tweed recorder directional data for about 20 metres water depth offshore from Letitia Spit. The wave height, period and direction data for the more intense May 1996 period, expressed as equivalent deep water values are presented in Figure 4.14.

Correlation is good between these data sets for height and period. Substantial differences are evident in the directions. These are affected at the Tweed recorder location by refraction from deep water, and have been converted to deep water equivalent values in the plotted results, based on the refraction model. Wave directions during the first of the storm period was predominantly from about east-northeast. Waves during the second, less severe event appear to have been somewhat more from the east to southeast sector. There appears to be a bias in the hindcast directions away from the east sector and towards the south to southeast. That pattern has been noted earlier in this document.

Longshore Transport

Longshore sand transport rates during these events have been derived from the modelling using the recorded wave data with BMO hindcast direction, except for the May 1996 storm for which the recorded Tweed data was used. As outlined above, transport rates calculated for such extreme events are likely to contain a potentially high error margin and should be regarded as indicative only.

The net transports rates obtained for each of these storms are summarised in Table 4.2.4. It can be seen that a relatively high proportion of the average annual net longshore transport may occur during these isolated short term events. The total transport in any year may be highly dependent on occurrences of such events. The bypassing system must be designed to cater for the associated high transport rates.

Table 4.7 Longshore Transport (m3/storm)

Storm Event	Net Longshore Transport at Location Shown (m^3/storm)				
	Kirra	Snapper Rocks	Point Danger (Frog's Beach)	Duranbah	Letitia Spit
April, 1989	+225,000	+254,000	+220,000	+155,000	+81,000
March, 1992	+86,000	+173,000	+274,000	+218,000	+206,000
February, 1990	+89,000	+110,000	+113,000	+70,000	+60,000
March, 1993	+120,000	+201,000	>+400,000*	+294,000	+301,000
February, 1995	+270,000	+251,000	+206,000	+154,000	+66,000
February, 1996	+12,000	+28,000	+84,000	+81,000	+97,000
May, 1996	>+400,000*	>+400,000*	+307,000	+193,000	-53,000

*Estimate - actual result affected by bed scour not modelled.

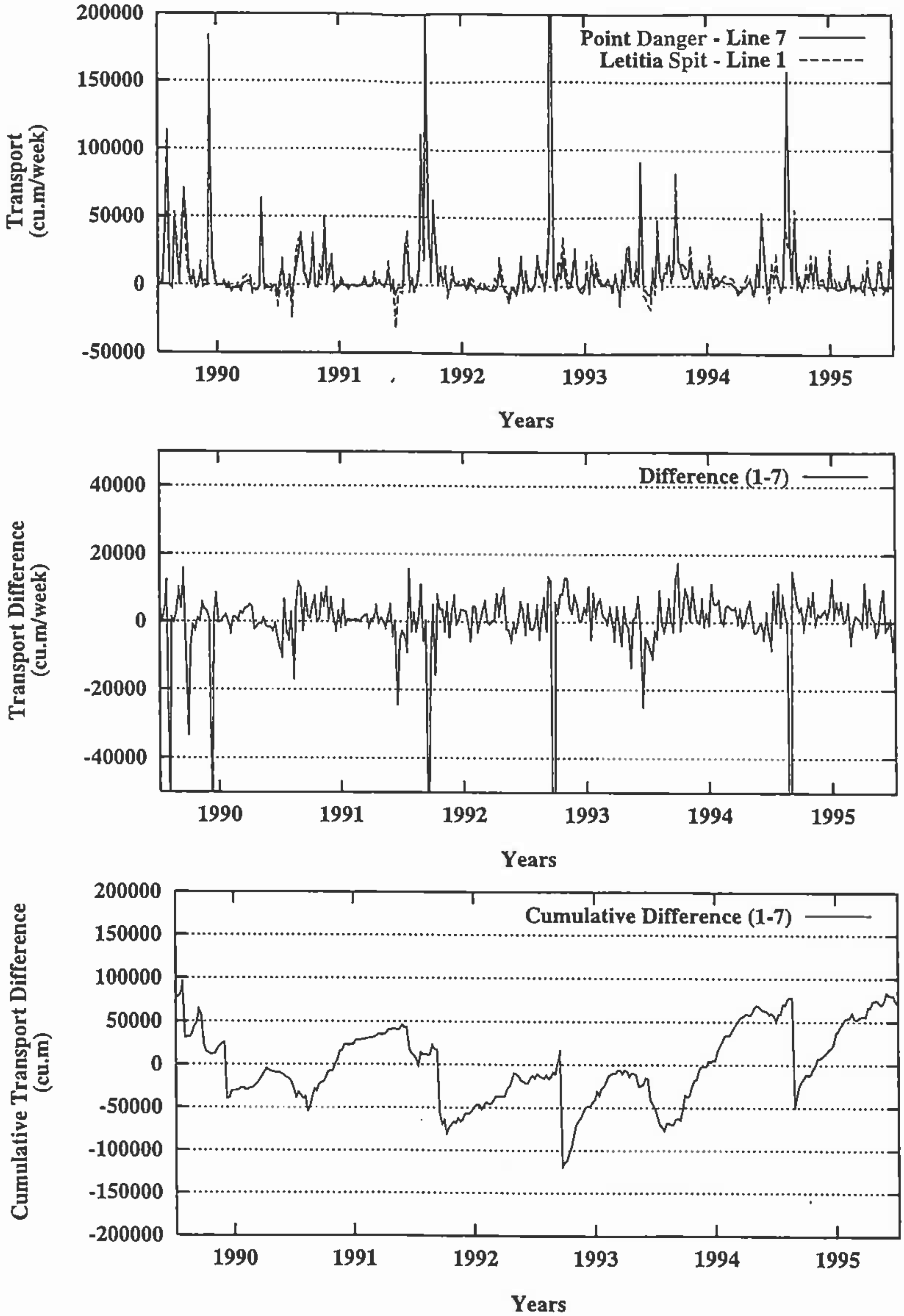


Figure 4.11
Longshore Transport Differential - Letitia Spit to Point Danger

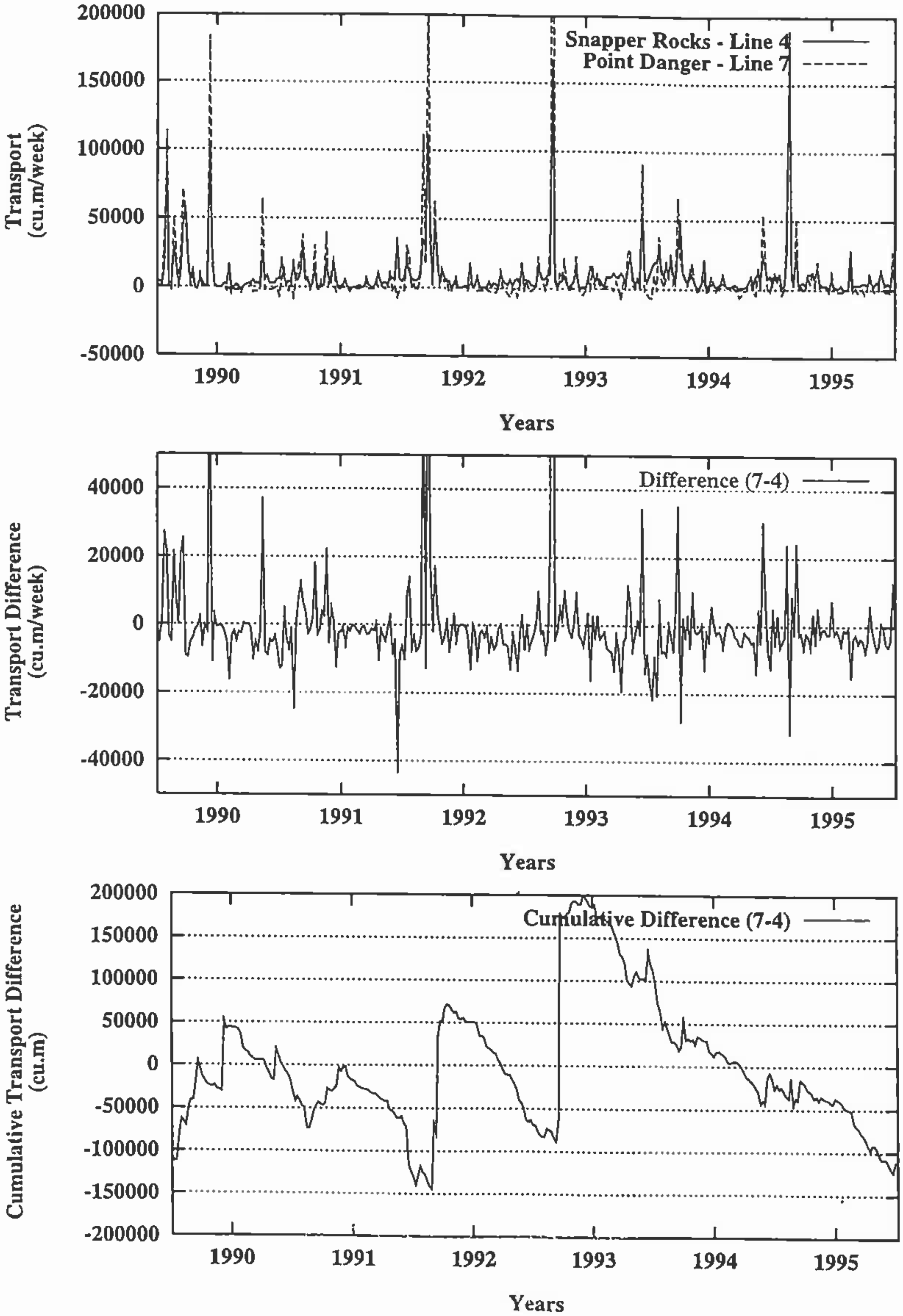


Figure 4.12
Longshore Transport Differential
- Point Danger to Snapper Rocks

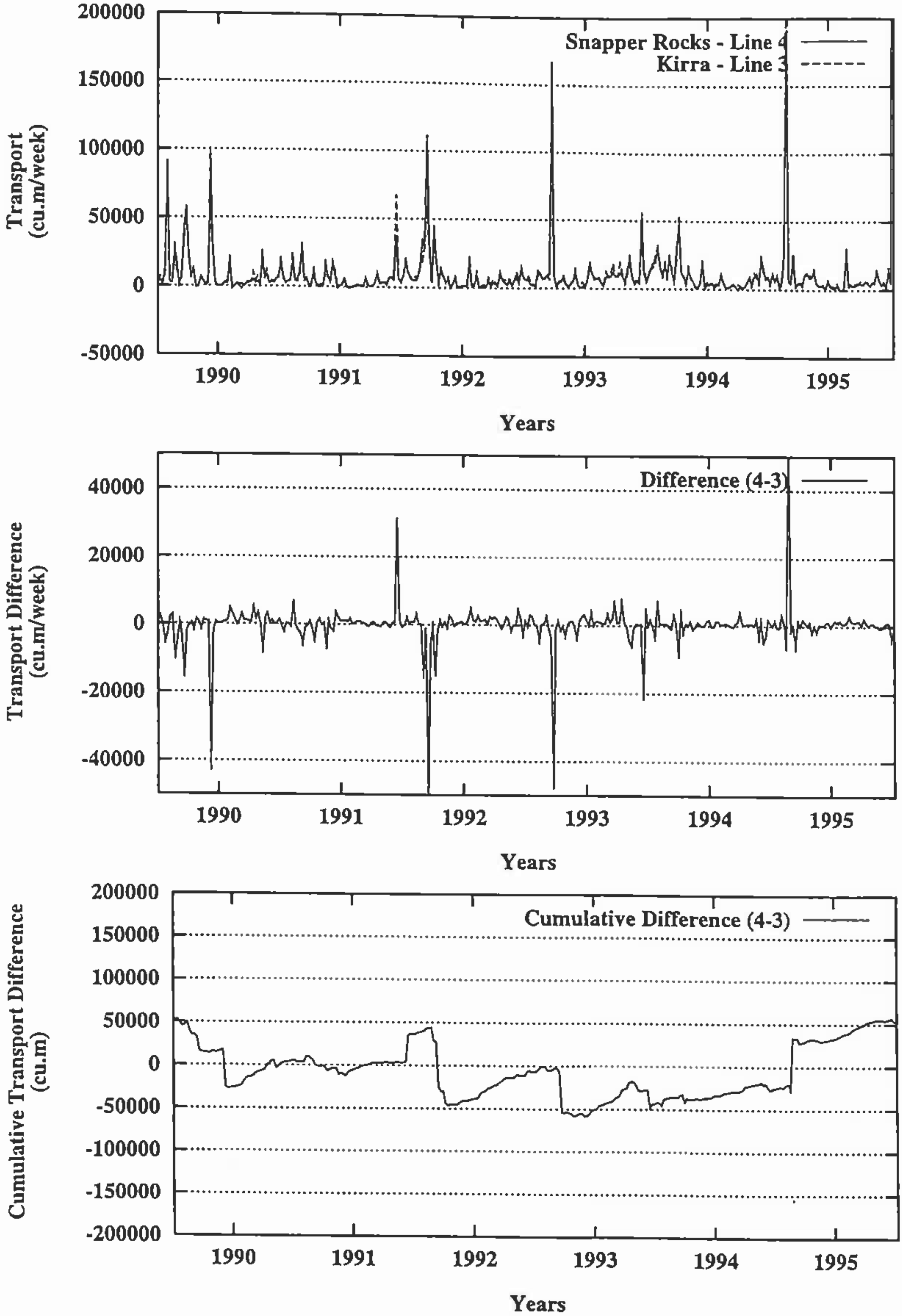


Figure 4.13
Longshore Transport Differential
- Snapper Rocks to Kirra

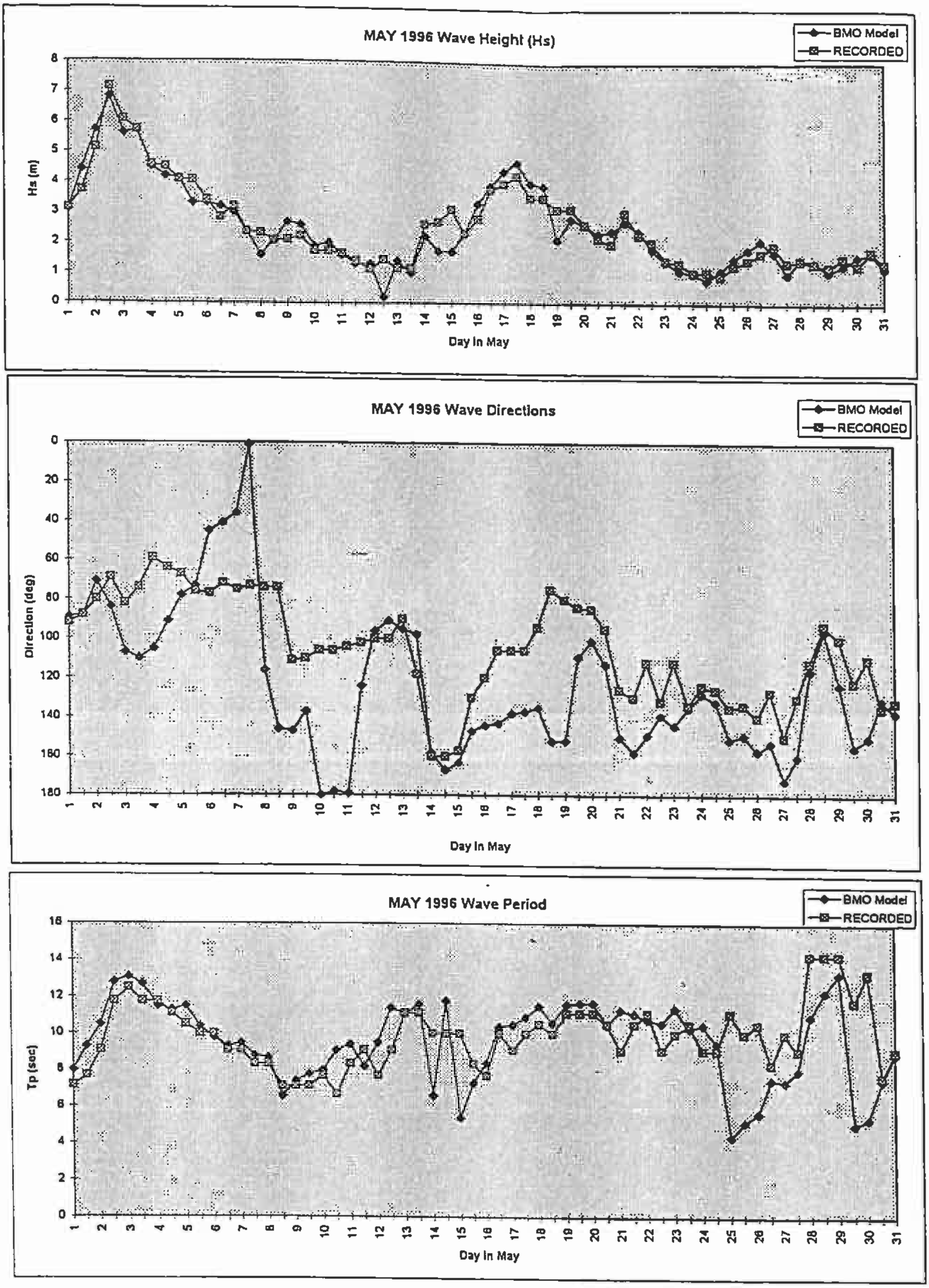


Figure 4.14
Wave Data During May 1996

5 Cross-Shore Sand Transport

5.1 General Considerations

Sand is transported across the nearshore beach profile by wave action. This cross-shore transport may be in the offshore direction during storm-related erosion events or onshore as the upper profile and beach recovers during normal swell conditions.

Cross-shore transport takes place essentially along the direction of wave propagation. It is a micro-scale transport process related largely to the effects of wave orbital velocities and induced net shore-normal current, particularly the surfzone bottom return flow.

Outside the breaker zone, particularly in deeper water, the oscillatory motion of the water, induced by wave action, provides the predominant forces on the bottom sediment. It is probable that bed-load dominates, with wave asymmetry providing a means for landward sediment migration due to higher (though shorter) crest velocities. The effect of gravity, through a bottom slope, opposes this shoreward migration.

However, bottom ripples are nearly always found in depths where sediment can be transported, and the situation becomes rather more complex. The formation of eddies within the ripple system causes sediment to be lifted more readily into suspension. While the larger grains are quickly deposited again following forward movement, the finer particles may travel some distance in either direction. Hence, a sorting action takes place with coarse grains migrating shoreward and finer grains moving seaward in suspension.

In instances where a general current, such as that due to the tide, wind, or return flow exists, the combined influence of all factors may give rise to net transport in the direction of that current rather than that of the net orbital movement.

Within the breaker zone, the return flow is relatively significant, and may carry considerable suspended material seawards. The breaker type is of importance, since it will influence the amount of turbulence generated near the bottom and the quantity of sediment in suspension.

In the wave run-up region, sediment may be deposited or eroded, depending on the foreshore slope, the grain size, the wave parameters and the water table level in the beach. The latter will control the amount of percolation of water into the beach, and thus the backrush quantities.

Overall, the following general pattern of sediment transport in both the offshore and surfzone regions has been recognised by various researchers:

(i) shorewards outside the breaker zone;

(ii) shorewards or seawards inside the breaker zone, depending on the balance between the net orbital velocity influence and the net return flow.

Both wave height and period influence the magnitude of the near bed orbital velocities. Hence, those parameters will determine the scale of the active profile, and the depth to which sand transport may occur.

Of considerable importance, particularly in shallower water, is the vertical distribution of net wave-induced mass transport. Even if there is no general transport of water in one direction (continuity conserved) there will generally be a seaward net transport in the middle and lower depths. This is readily understood by considering the fact that, for waves of finite height, the same quantity of water must flow seaward under the trough (smaller depth) as flowed landward under the crest (greater depth). The seaward return flow may carry sediment in suspension into somewhat deeper water.

Within the breaker zone, wave breaking with spilling or plunging of the crest, entrapment of air in the water, and great increase in the general turbulence, will cause much suspension of sediment. Here even larger grains agitated into suspension by the orbital velocities will travel quite some distance under the influence of existing net currents.

It is important to bear in mind that the breaker zone is the area in which the vast majority of the energy of the waves is dissipated. This can be achieved only by:

(a) movement of sediment;

(b) conversion to other forms such as noise;

(c) reflection, either in direct wave form or as a current; and

(d) conversion to potential energy and hence to flow in other directions.

The relative magnitudes of each of these mechanisms is determined by the wave, water, and sediment properties, and in turn determines the sand transport pattern.

In shallow water, and on the foreshore, the wave period plays an important role in determining the phase lag between wave uprush and backwash, in conjunction with wave height and foreshore slope. This phase lag, discussed at length by Kemp [9], apparently is important in governing the erosion or accretion tendency.

The grain size of the beach sand will affect the ability of the flow velocities to move the sand. The depth to which transport occurs, and to which a pre-existing profile can be modified, varies inversely with grain size. It has been identified also that the foreshore slope is strongly related to grain size, increasing as the size increases.

Coarser sediment tends to migrate shorewards due to the sorting action of asymmetric waves. This may be the case even if no finer particles are available for offshore transport in suspension. Hence coarse grained beaches exhibit accretion features with rather steep nearshore profile slopes. Sediment mobility at any depth varies inversely with grain size, as does (presumably) the rate of profile modification at that depth.

In prototype and in model tests, the tendency of the beach profile towards characteristic shapes determined by the wave and sediment properties is well documented. The existence of profiles of equilibrium, in harmony with the waves, appears to have been verified.

Bakker (1968) proposed that onshore-offshore transport at any time is proportional to the difference between the equilibrium profile form and the actual profile form at that time.

However, the continuing changes in wave conditions usually do not permit profile modification to reach the equilibrium state. Nevertheless, the profile may reach a dynamic equilibrium with seasonal or averages wave conditions over a long period of time. During storms of limited duration, equilibrium may not be achieved, particularly outside the breaker zone, although very large quantities of sediment can be moved within the breaker zone region.

The deeper parts of the profile, while subject to sediment transport during storms, remain undisturbed by the smaller "normal" waves. Over a period of many centuries, assuming little water level and climate variation, these deeper parts may become permanently in equilibrium with the storm condition. However, a transition from the active part of the profile to the region of no movement exists.

It has been recognised that high values of wave steepness (H_o/L_o) will lead to beach erosion, often with the formation of offshore submerged bars. Swart (1974) documents offshore transport rates of 50-80m^3/m/hr in North Holland during the 1953 storm.

5.2 Measured Nearshore Profile Variations

A selection of surveyed profiles have been extracted from the BPA database for further analysis. The survey lines reviewed are:

Letitia Spit:	ETA 4.0 ETA 6.0 ETA 8.0 ETA 10.0
Duranbah:	ETA 12.0 OMEGA 9.0
Point Danger:	RB 1.0 RB 2.0
Rainbow Bay:	RB 3.0 RB 4.0
Greenmount:	GREEN 1.0
Coolangatta:	CG 3.1 CG 6.0
Kirra:	K 1.0 K 10.0 ETA 14.0

The locations of these survey lines are shown on Figure 5.1. All available and relevant surveys for the above profiles are shown in Figures 5.2 (a)-(d). General observations are as follows:

- Letitia Spit

  The Letitia Spit profiles show considerable variability in shoreline location, the variability increasing northward towards the Tweed River entrance. There exists an oscillation in the shoreline location in response to storm erosion etc on top of a general progression of the shoreline seaward. The seaward progression is due in most part to the beach line response to the construction of the entrance walls to the Tweed River. The storm induced variability of the profiles extends to a depth of 12-15 metres with single offshore bars of crest level at about R.L -4 to 6 metres AHD developing about 300-500 metres offshore.

- Duranbah to Point Danger

  Duranbah shows considerable variability. This has occurred as a response to the construction of the Tweed River entrance walls. The deepwater (ETA 12.0) profile has evolved from a slope similar that south of the Tweed River entrance (1:80) to a much steeper slope similar to those off Snapper Rocks (1:40). Storm related profile variations exist to a depth 10-12 metres.

At Point Danger, the nearshore profile has recovered from an eroded depth of about 6 metres to about 3 metres as a result of renewed bypassing. Storm wave activity creates a bar and gutter at chainage 900-1200m (4-10m depth) via cross-shore (offshore) and longshore transport mechanisms.

- Snapper Rocks to Coolangatta

  The profiles become less steep moving from Snapper Rocks (RB 3.0, 1:30) westwards to Coolangatta (CG 3.1, 1:100). Short and longer term variability in the profiles to a depth of 10-12 metres is apparent at RB 3.0 and RB 4.0 while the profiles at GREEN 1.0 and CG 3.1 show little short term variation below a depth of about 5 metres. Also of note is the recent offshore nourishment indicated by the large bars in profiles GREEN 1.0 (chainage 600-800m) and CG 3.1 (chainage 600-1000m).

- Coolangatta to Kirra

  This area shows wide gently sloping (1:100) profiles. Significant longer term variations in the profiles at the shoreline are apparent. This is due to long term erosion caused by the training nourishment exercises at Kirra and offshore in October 1988 and 1989. Recent offshore nourishment can be seen by the presence of bars at CG 6.0 (chainage 600-1000m), K 1.0 (chainage 800m) and K 10.0 (chainage 850m). ETA 14.0 profiles were surveyed from 1966 to 1983 and indicate that profile changes by wave/current action extend out to a depth of 12-15 metres, in this case erosion due largely to the lack of incoming sand as a result of the construction of the Tweed River training walls.

It is generally accepted that, during large storm events, sand transport occurs to a depth of about 12-15m. This is reflected in the variability of the Letitia Spit profiles to this depth. The wider more gently sloping Coolangatta and Kirra beach profiles experience transport to these depths as indicated by the profile modifications at ETA 14.0. However, recent profiles at Coolangatta to Kirra (CG 6.0, K1.0, K10.0) do not show any significant changes below about 7 metres. This lack of modification indicates that these profiles are in an equilibrium.

This is an important observation because many storms have occurred over the survey period yet no significant changes in the Coolangatta to Kirra profiles below 7m are recorded. Processes that contribute to the observed equilibrium are identified as follows:

- The area is subjected to reduced wave heights due to refraction and bed friction affects around Snapper Rocks and across the profiles and there is a corresponding lack of influence of wave turbulence at greater depths (12-15m).
- It is likely that, due to the gentle slope, the transport due to storm waves occurs over a wide zone and is without a well defined break point. Thus, the intensive action of the waves over a narrow zone, resulting in bar and gutter formations, does not occur.
- Bars and gutters form due to cross-shore transport associated with wave action when the wave approach angle is approximately shore normal. Storm waves approaching from the SE to E attack the Letitia Spit beaches at (or near) shore normal and cross shore transport mechanisms tend to dominate and form of bar systems. Similar storm waves approach the Coolangatta and Kirra profiles at an angle and cross shore related transport (in the direction of the wave) will occur at an angle (in some cases almost parallel) to the shoreline. In such cases offshore bars and gutters will not form and the component of sand transport due to wave action towards the shoreline is small.

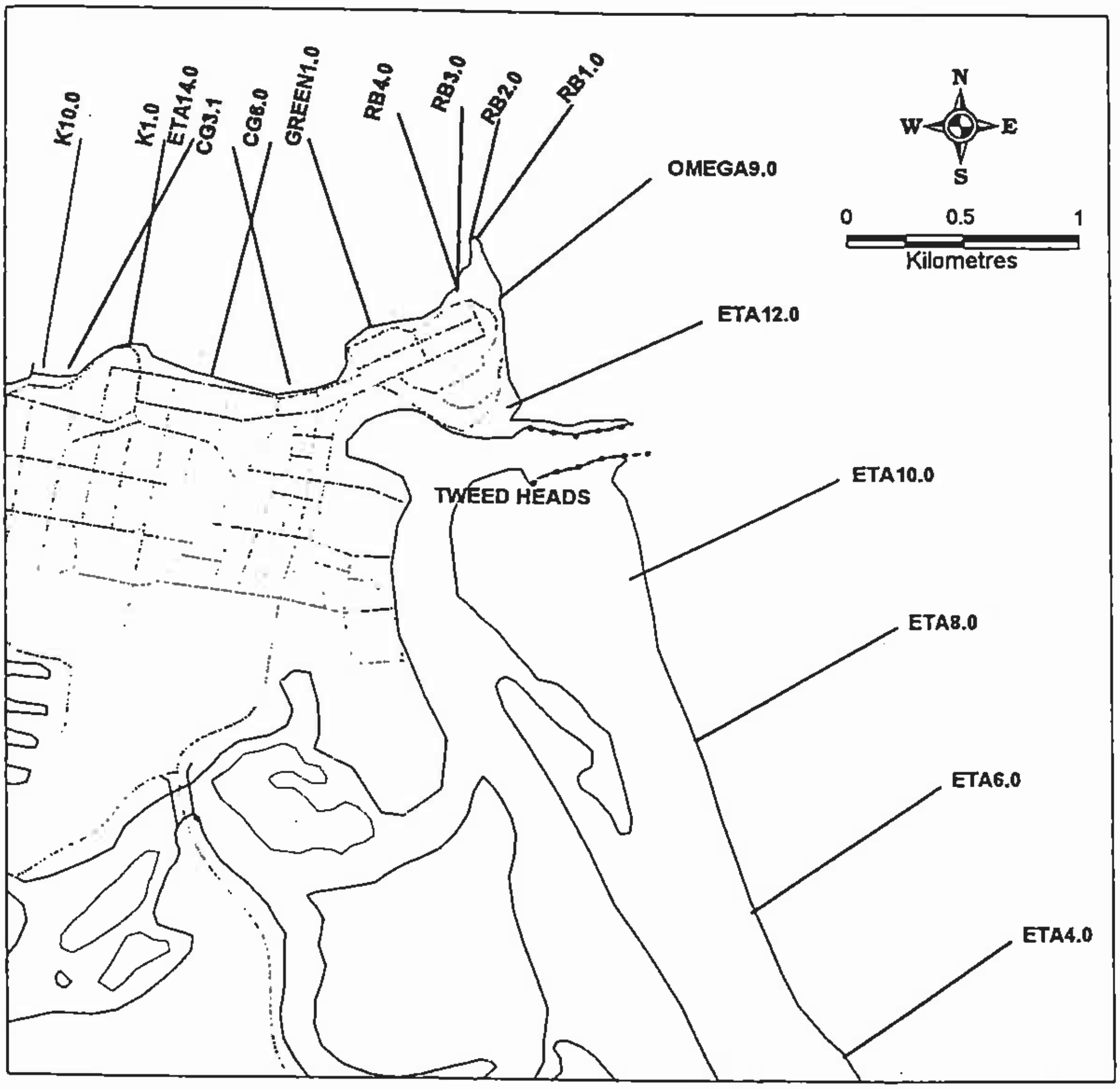


Figure 5.1
Location of Profile Survey Lines

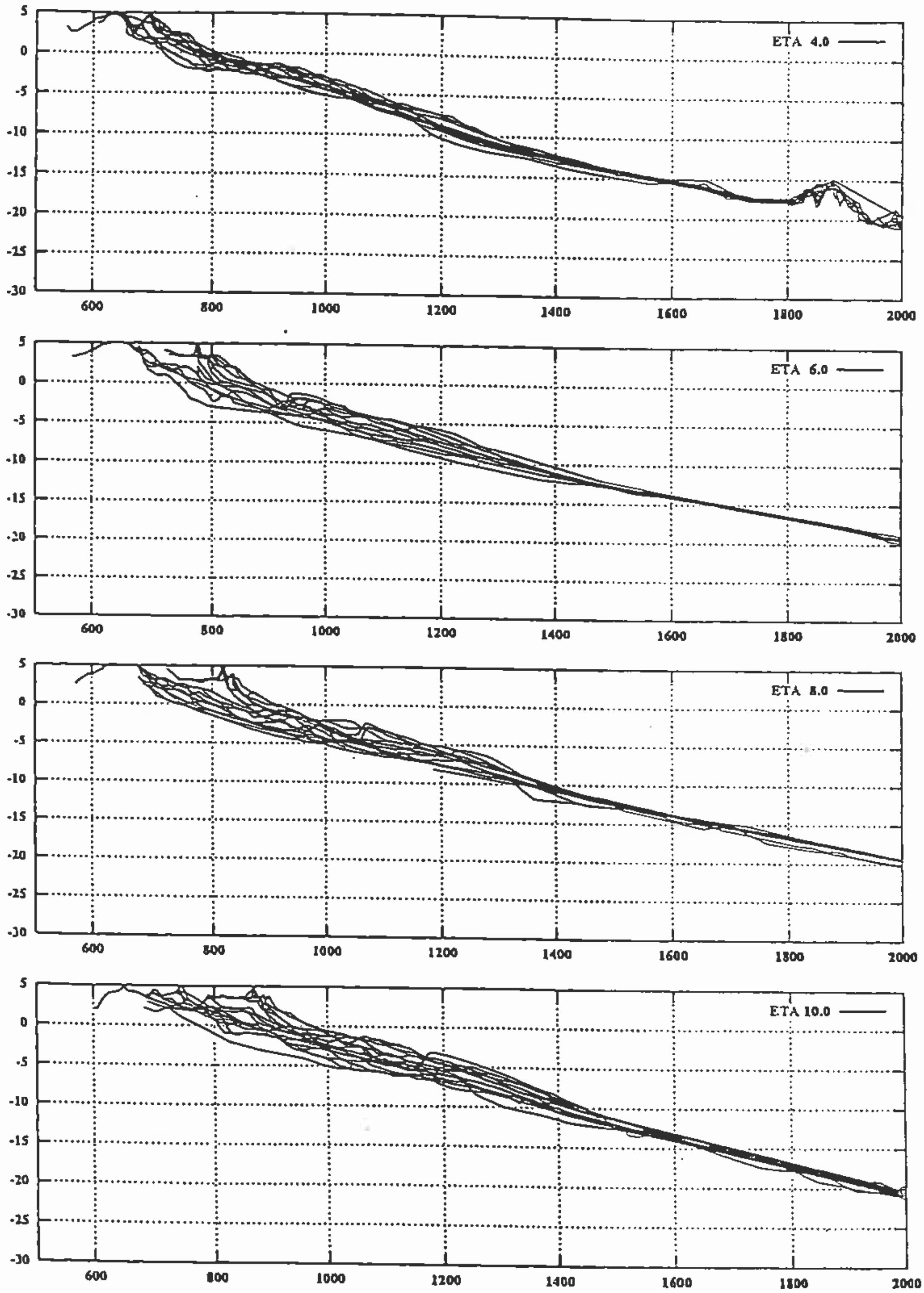


Figure 5.2a
Nearshore Profile Surveys - ETA 4.0 to ETA 10.0

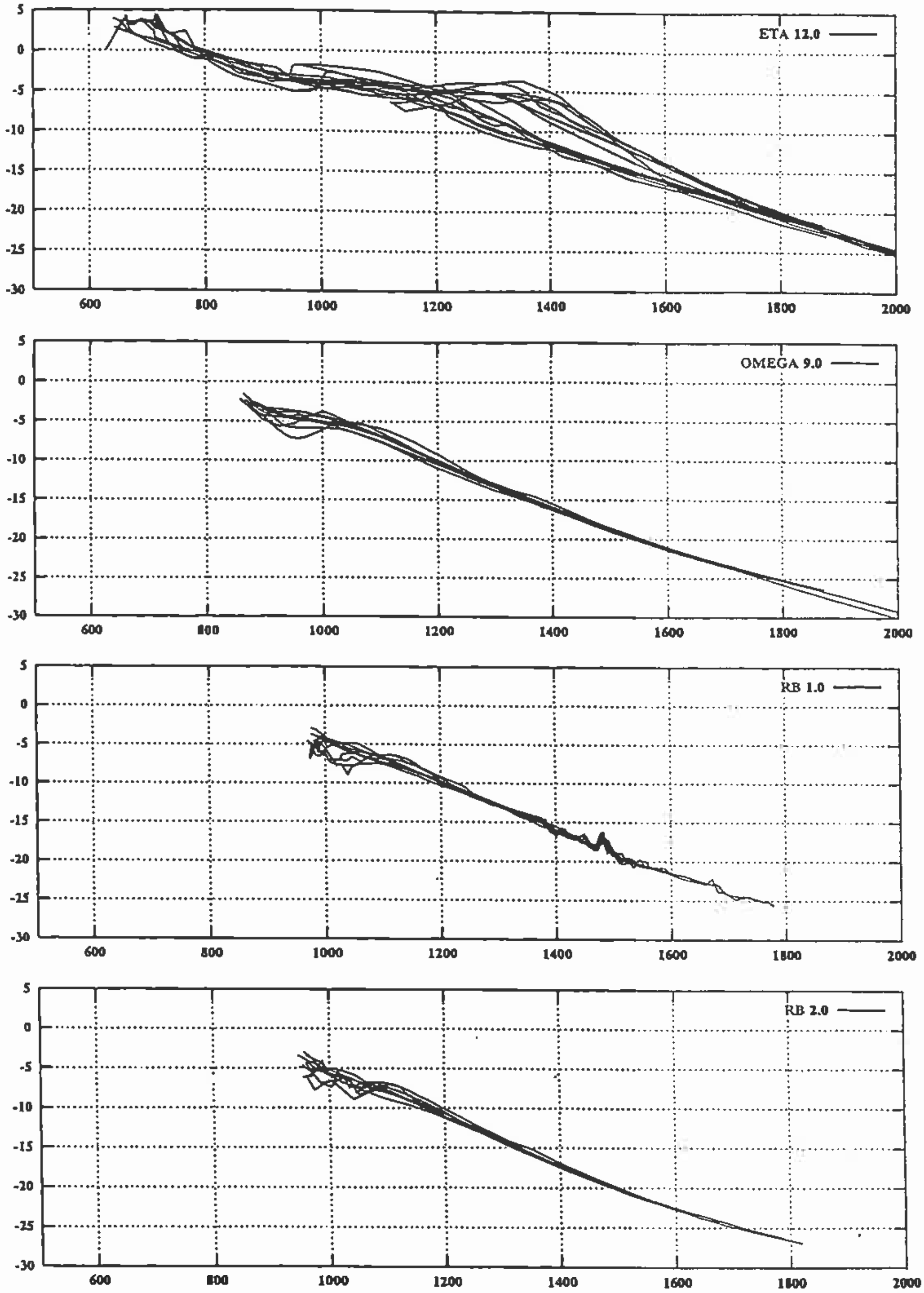


Figure 5.2b
Nearshore Profile Surveys - ETA 12.0 to CG 3.1

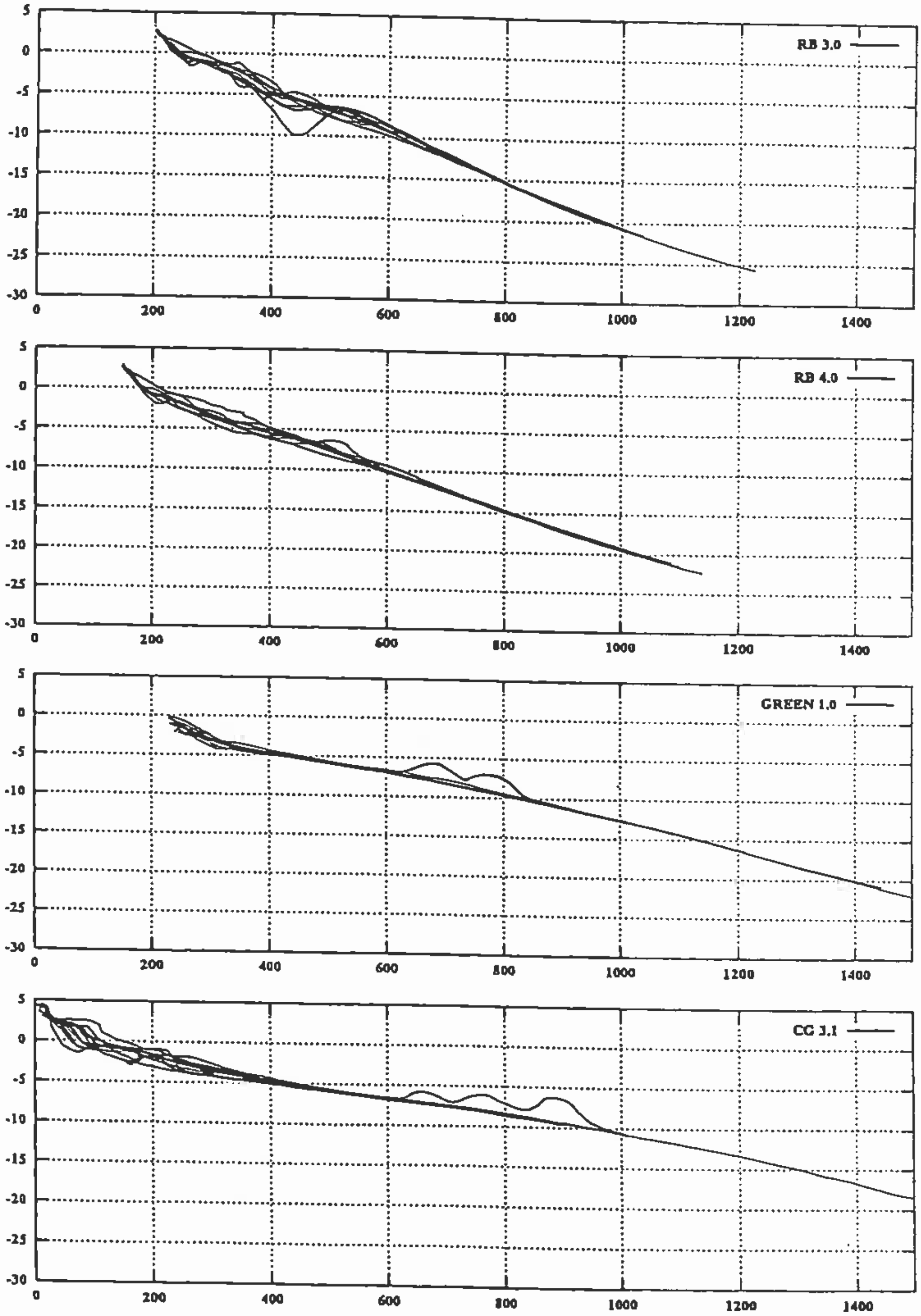


Figure 5.2c
Nearshore Profile Surveys - RB 3.0 to CG 3.1

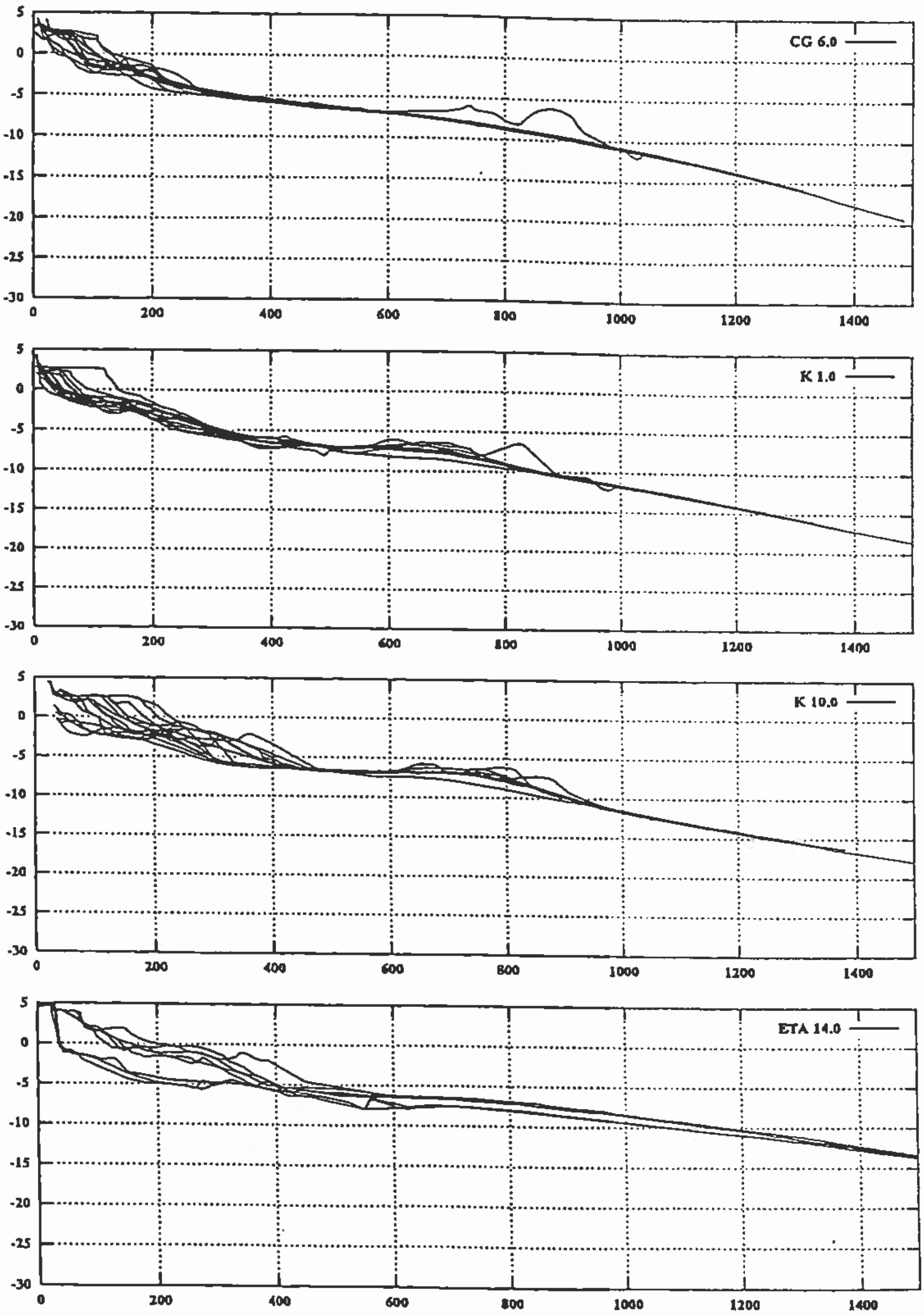


Figure 5.2d
Nearshore Profile Surveys - CG 6.0 to ETA 14.0

5.3 Measured Cross Shore Transport Rates

An estimate of cross shore transport rates for beaches in the Tweed and Southern Gold Coast regions can be determined from investigation of selected survey profiles. Of interest are profiles where appropriate surveys where the following processes have occurred

- post storm recovery of an offshore bar
- the generation of a storm related offshore bar
- onshore transport of offshore nourishment

The data examined shows few occasions where detailed surveys close to a storm event allow determination of cross shore transport rates because generally the time between the event and survey is too long. Cross shore transport theory indicates that profile change is most rapid when the profile is furthermost from the equilibrium shape. Therefore, to be of greatest benefit in determining the effects of a particular storm, surveys need to be undertaken immediately (1-2 days) following the event.

In an attempt to quantify probable cross shore transports rates an assessment of survey ETA 63.0 has been undertaken as data from numerous survey events are available. This survey line extends eastwards from Surfers Paradise. The line has similar geographic characteristics to those on Letitia Spit being a relatively long straight eastwards facing beach. Surveys during 1974 and 1988 have been reviewed and selected surveys are illustrated in Figures 5.3 (a)-(b). All show surveys documenting an onshore transport of sand from a storm bar at a depth of about 4 metres. The rates of transport for each have been calculated as follows

- 04/88-05/88 820-1100 m^3/m/yr (nearshore ~ 450 m^3/m/yr)
- 07/88-08/88 240-430 m^3/m/yr
- 08/88-09/88 920-1200 m^3/m/yr (nearshore ~ 310 m^3/m/yr)

Surveys at ETA 75.0 (offshore from The Spit) show the onshore migration of a bar as illustrated in Figure 5.4. The transport has occurred in much deeper water than at ETA 63.0 and the transport rates are somewhat less, averaged over three months as follows:

- 07/67-10/68 165 m^3/m/yr

Patterson (1976) carried out a detailed analysis of this data in terms of both depth and seabed slope. His results for onshore transport are presented in Figure 5.5. They are consistent with the above rates.

The above rates relate to onshore transport by non-storm wave conditions. Offshore transport is usually associated with larger waves generated during storm events. Unfortunately, few surveys sufficiently document a "before and after" profile definition associated with a storm event. Typically, the storm induced cross and longshore transports are of such a magnitude and complexity that it is difficult to identify the individual processes for a given survey line.

An indication of offshore transport rates associated with such storm events is therefore difficult to quantify. A review of ETA 63.0 profiles against idealised equilibrium profiles suggests that offshore storm related transport rates could be of the order 60-160 m^3/m/day for the duration of the storm (several days), but with significantly higher rates almost certainly occurring for a limited period during the peak of the storm.

Cyclone Erosion

The data examined shows few occasions where detailed surveys close to a storm event allow determination of cross shore transport rates. Generally the time between the event and survey is too long. Cross shore transport theory indicates that profile change is most rapid when the profile is furthermost from the equilibrium shape. Therefore, to be of greatest benefit in determining the effects of a particular storm, surveys need to be undertaken immediately (1-2 days) following the event.

Considerable data related to extreme cyclone erosion was measured along the Gold Coast beaches in 1967. Pre cyclone conditions were well documented during 1966. Massive erosion of all beaches occurred in July 1967. Comprehensive surveys of the erosion were undertaken in August/September 1967.

These data indicate the following potential offshore transport quantities from the beach, dune and upper nearshore profile, typically out to about the RL-3m(AHD) depth contour.

Location	Erosion Quantity (m^3/m)
Coolangatta	415
Palm Beach	153
Surfers Paradise	387
The Spit	351

The abnormally high erosion quantity at Coolangatta extends out to the RL-6m(AHD) depth contour and reflects the impact of the Tweed River training walls in denuding the nearshore zone, thus causing increased demand of sand from the beach, dune and upper nearshore profile.

Little more recent data for specific storm events is available. It is expected that, given the severity of the 1967 erosion, the above quantities represent an upper limit to design conditions.

Beach widths vary substantially associated with storm and cyclone erosion. Moderate storm events may cause shoreline retreat of 20-40 metres. More extreme erosion may occur less frequently, with shoreline retreat of 50-80 metres and extensive potential dune erosion part of the natural behaviour of these beaches.

5.4 Model Provisions

5.4.1 Wave Induced Transport

Modelling software has been developed to simulate cross shore transport taking into consideration processes due to the following

- wave shoaling;
- wave setup;
- tide;
- wave orbital velocities;
- surfzone bottom return flow; and
- instantaneous sand transport rates.

In developing this model the following assumptions have been made:

Figure 5.3a
Nearshore Profile Surveys: ETA 63 - 1974

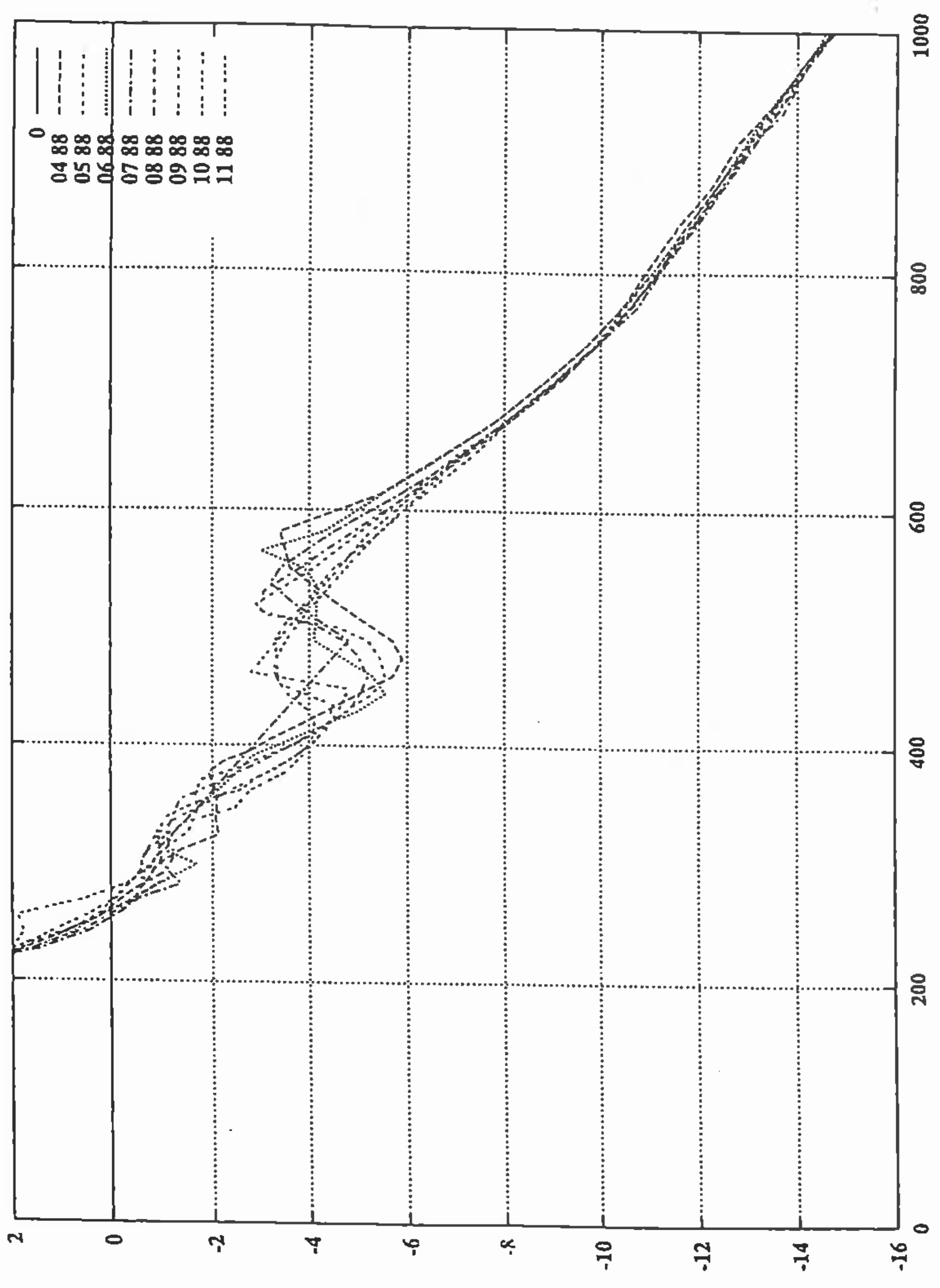


Figure 5.3b
Nearshore Profile Surveys: ETA 63 - 1988

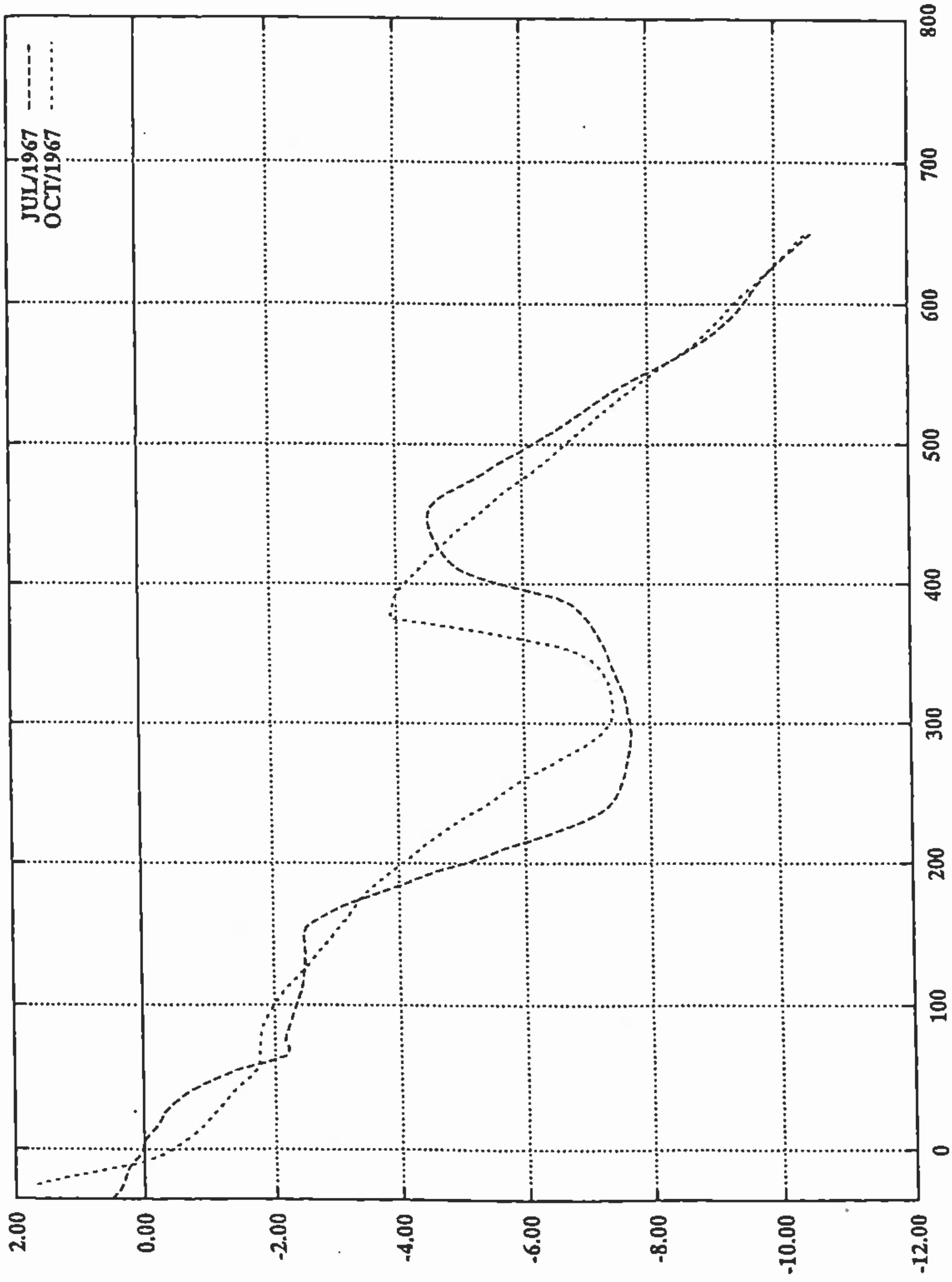


Figure 5.4
Post Cyclone Nearshore Profile Surveys:
ETA 75 - 1967

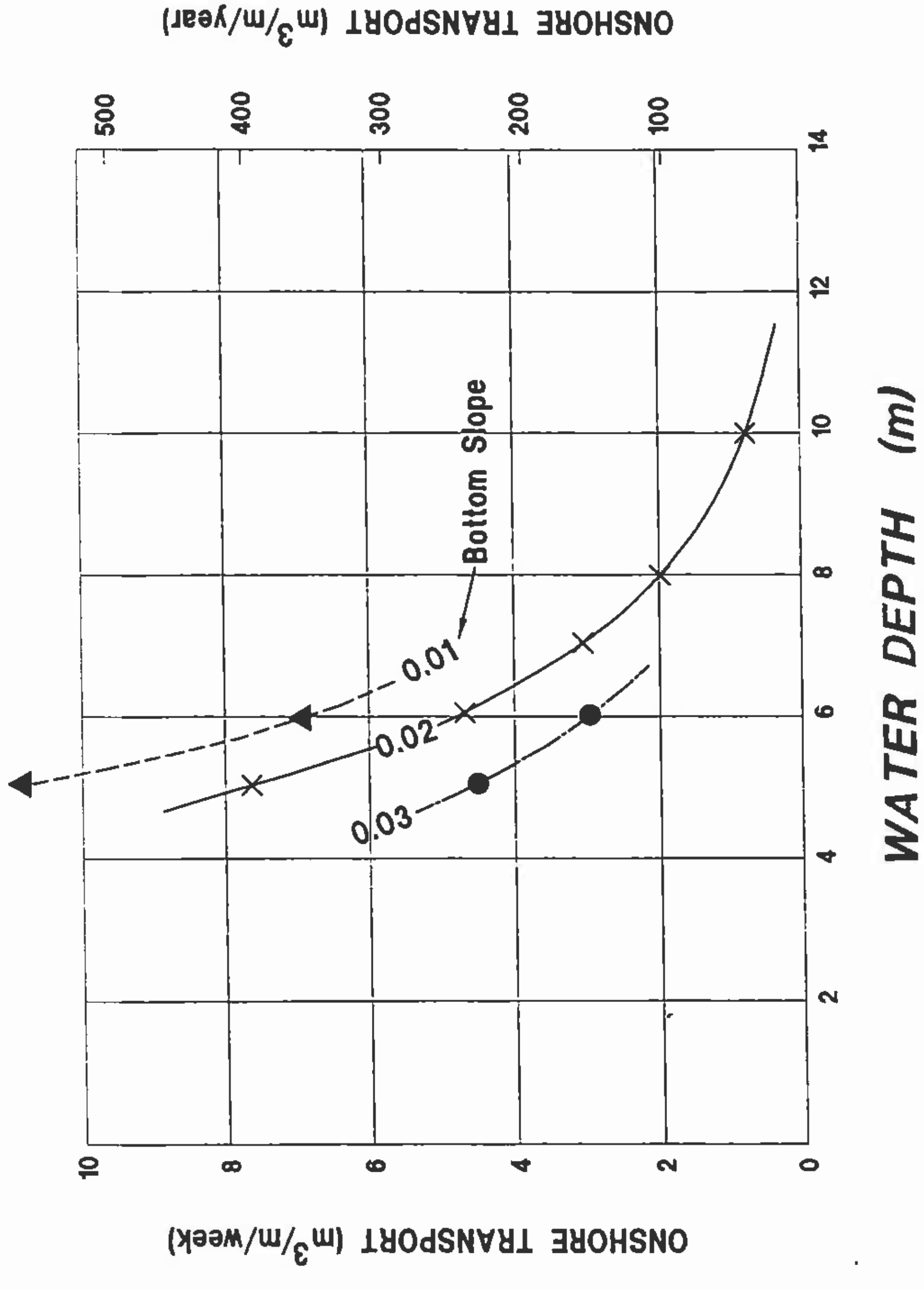


Figure 5.5
Measured Cross-shore Transport Rates - ETA 75

- longshore transport for a given profile is constant (ie bar growth and decay are due to on/off shore processes only);
- the cross-shore profile is defined along the wave orthogonal; and
- irregular waves are approximated by the Rayleigh distribution of wave heights.

The method of Bagnold-Bailard 1981 (as presented in Van Rijn 1990) for the calculation of cross shore transport rates has been adopted, with some modifications to facilitate practical representation of the cross-shore transport processes. In this formulation the instantaneous bed load and suspended load transport rates are calculated.

The following describes the generalised methodology used in the model for the calculation of cross shore transport rates:

1. The deepwater incident wave condition is used to define a distribution of random waves defined by the Rayleigh distribution. The distribution of waves propagates along the profile, shoals, breaks and decays to the shore (secondary shoaling or wave reformation may occur).
2. An energy distribution across the beach profile due to the shoaling and breaking characteristics of the random waves is calculated based on the Rayleigh distribution, and used to determine an equivalent wave height profile and wave setup.
3. Asymmetric near bed orbital velocities are calculated from the equivalent wave height distribution and modified to incorporate a return flow component (see below).
4. Incremental transport rates are determined and summed over the wave period.

5.4.2 Return Flow

The energy flux towards shore for a given wave is constant until the wave begins to break and decays towards shore. As a wave shoals and breaks towards shore, a mass of water in the broken part of the wave crest is moved shoreward. The return flow, or undertow (in conjunction with wave setup) provides an energy/mass balance to this process.

As a wave breaks and decays in height towards the shore, the energy flux is no longer constant. In this model, the return flow is set proportional to the energy flux losses for the breaking wave.

5.5 Validation of Modelling Approach

Surveyed cross-shore profiles, one on Main Beach (ETA 75) and one near Surfers Paradise (ETA 63) and a range of sections within the study area which have been regularly surveyed since 1966, have been used both to determine cross-shore transport rates and to calibrate this model.

For onshore transport in deeper water, cross shore transport simulations were performed with a start point being the eroded bar profile of July 1967. While no wave data is available for that time, typical July-November data was used. Model results show that the crest of the bar moves shoreward a similar distance to that of the observed data (approximately 80m) during this period with an assessed onshore transport rate in about 6 metres water depth of 150-180m^3/m/year.

As well, onshore transport rates were modelled for typical Gold Coast wave conditions (a year of data for 1990) for several plane profile slopes and transport rates obtained at a range of depths. The results are presented in Figure 5.6. They show good correlation with the measured results and those of Patterson (1976).

5.6 Modelled Cross-shore Transport Results

The modelling program developed for determining cross-shore transport rates has been applied at each of the calculation locations through the study area. Appropriate wave refraction coefficients have been applied in each case to simulate the wave climate along the wave orthogonals.

Results of these computations are presented in Appendix D for the following profiles:

- CG 3.1 Coolangatta
- ETA 8 Letitia Spit
- ETA 12 Duranbah
- K 10 Kirra
- RB 1 Snapper Rocks
- RB 4 Rainbow Bay

Transport rates for each profile are presented as:

(i) Time Series: At selected locations in each profile time series of transport rates are presented for 1990 wave climate. The daily transport rates (dotted) and a moving average of these (solid) are presented.

(ii) % Occurrence: At these locations, the % occurrence of transport rates onshore (+ve) and offshore (-ve) are presented for 1990 - 1992 wave climate.

Results from the cross shore simulations show the high temporal variability in transport at each depth in the profile. This is, of course, related to the influences of wave height and period at that location. Transport below a depth of about 8m is small (less than $1m^3/m/day$) and sporadic, related to larger wave events.

Generally, onshore and offshore processes occur to depths of about 4m, while beyond these depths the dominant process is onshore transport. During larger wave events offshore processes can occur to depths of 8 - 10m.

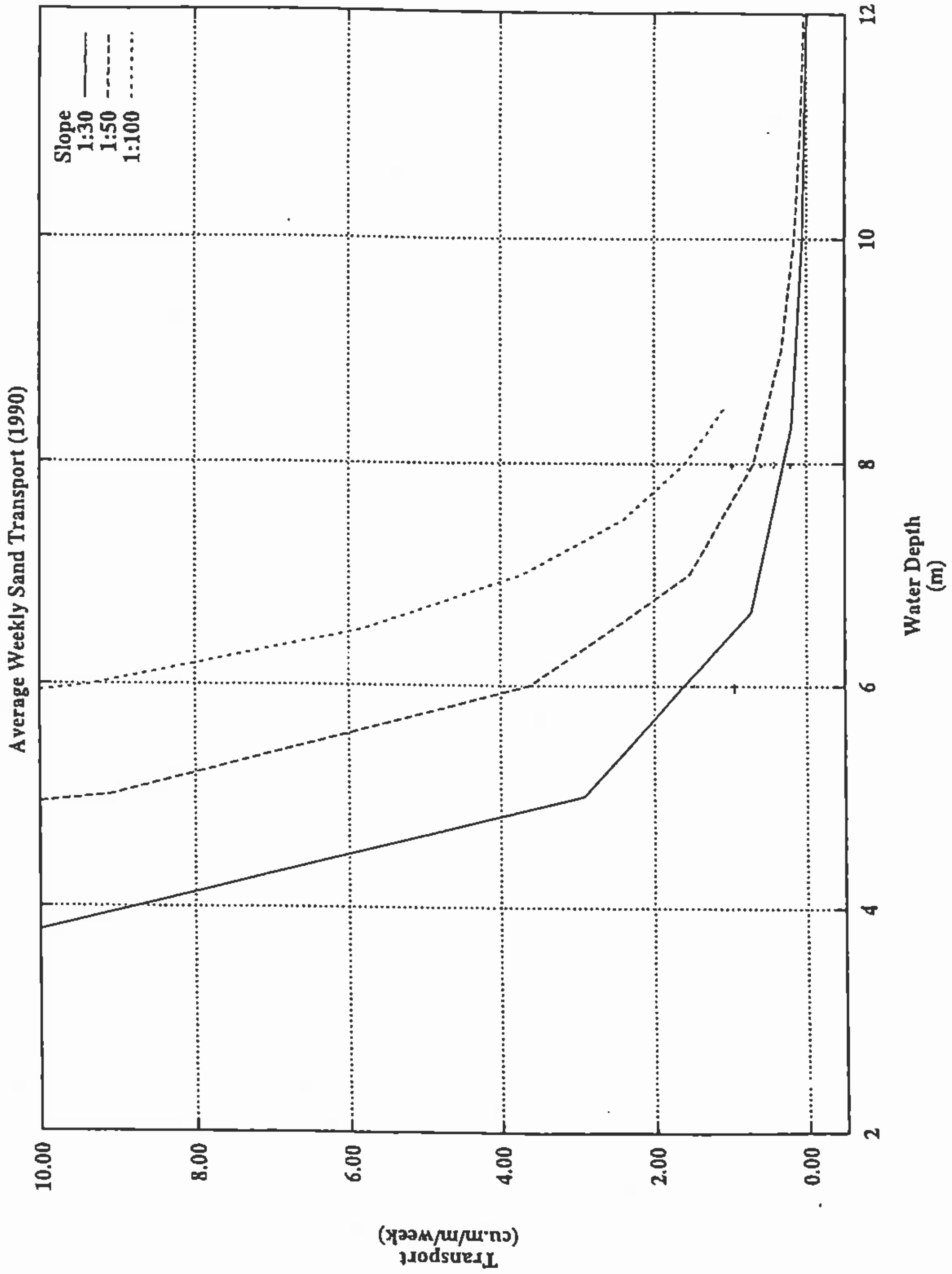


Figure 5.6
Modelled Cross-shore Transport Rates

6 Assessment of Beach System Response to the Sand Bypassing

6.1 General Considerations

Assessment of the response of the beach system to the proposed bypassing has involved both shoreline evolution modelling and application of other historical and two dimensional modelling information.

For shoreline evolution modelling to assess the variability of the sub-aerial part of the beach system, the software package GENESIS developed by the US Army Corps of Engineers (CERC) has been used. This utilises the conventional CERC equation for longshore transport. While not capable of modelling the complex two-dimensional coastal processes of this area, it is useful for estimation of beach responses to natural wave variations and the proposed bypassing works.

GENESIS is a program developed to simulate shoreline change associated with a wide range of coastal processes and works. The name GENESIS is an acronym that stands for GENEralized Model for Simulating Shoreline Change. GENESIS contains what is believed to be a reasonable balance between present capabilities to efficiently and accurately calculate longshore sediment processes from engineering data and the limitations in both the data and knowledge of sediment transport mechanisms and beach change. The modelling system and methodology for its use have matured through application to numerous types of projects, yet the framework of the system permits enhancements and capabilities to be added in the future.

6.2 Shoreline Modelling Methodology

GENESIS simulates shoreline change produced by spatial and temporal differences in longshore sand transport. Shoreline movement such as that produced by beach fills, river sediment discharges and sand extraction can also be represented. The main utility of the modeling system lies in simulating the response of the shoreline to structures and nourishment/extraction sited in the nearshore zone. Shoreline change produced by cross-shore sediment transport as associated with storms and seasonal variations in wave climate cannot be simulated. Such cross-shore processes are assumed to average out over a sufficiently long simulation interval or, in the case of a new project, be dominated by rapid changes in shoreline position from a non-equilibrium to an equilibrium configuration.

The modelling system is generalised in that it allows simulation of a wide variety of user-specified offshore wave inputs, initial beach configurations, coastal structures, and beach fills. It provides for wave propagation to the site by internal formulations or, in complex bathymetric areas and near structures, by linking directly with specialised wave propagation software (RCPWAVE) to obtain the nearshore wave field for longshore transport calculations.

GENSIS is ideally suited to beaches with relatively uniform shape and nearshore bathymetry. In more complex areas, its principal limitation is its 'one-line' schematisation of the shoreline and nearshore profile. That is, it assumes a constant nearshore profile shape which may change position as more or less sand is available in any beach increment. It does not provide for cross-shore transport, except in the form of sand fill, extraction or bypassing which have the effect of addition or removal of sand volume from the upper nearshore profile.

Thus, at a beach such as Duranbah where the northward extension of the entrance bar shoals carry a significant proportion of the longshore transport, the model can simulate only the beach processes in the upper profile area. Onshore supply of sand from the nearshore shoals can be represented as progressive beach fill.

Two GENESIS models have been used to simulate the beach units in the study area.

Model 1:
- a model facing East incorporating Letitia Spit, the Tweed Entrance and Snapper Rocks; and

Model 2:
- a model facing North incorporating the beaches from Snapper Rocks to North Kirra.

The model locations are shown in Figure 6.1. In both models, the waves were input in time series form, being a full year (January 1995 to January 1996) of recorded height period and direction data from the Tweed recorder, repeated as appropriate to represent multiple year shoreline evolution. While this does not represent the full range of potential wave conditions, it is sufficient to both indicate variability through the year of shoreline response to varying wave conditions and assess in principle the impact of the bypassing in improving the beaches.

The choice was made to use the internal wave transformation formulations for Model 1. This was considered acceptable because of the more regular offshore bathymetry. This combination allows the best resolution and the most efficient determination of the variability of the beaches in response to real wave conditions.

Because of the formulation GENESIS uses to determine the offshore profile the models are most suited to simulating the beach changes down to a depth of 6 to 10 metres. Therefore the upper nearshore and lower beach responses are best represented and any transport by deeper water bar related mechanisms may be incorporated as a bypassing component.

For Model 2, RCPWAVE propagation results were used to provide input wave condition at a nearshore location in about 6 metres water depth. The internal transformation formulations are used to transform the waves from that location to shore. The use of extended real wave timeseries data means that the natural response of the sediment supply and offshore storage areas to the pulsing of supply past Snapper Rocks could be simulated. The model was set up to approximate the shoreline with headlands represented as a combination of groynes and seawalls. The base case (1995) was established by allowing the shoreline to evolve to an equilibrium situation approximating that existing at that time with uniform average annual longshore transport through the model. A base profile depth of 6 metres was adopted such that only about 60% of the total longshore transport occurred within the nearshore zone represented in the model.

The effect of reducing the length of the Kirra Groyne and restoring the full sand supply were then simulated, the latter by modifying the updrift boundary to increase the sand supply rate through the beach system. The modelled longshore sand transport rates as derived by the GENESIS methodology were thus increased from a base rate of about 220000 cubic metres per year to about 300000 cubic metres per year within the nearshore zone represented in the model.

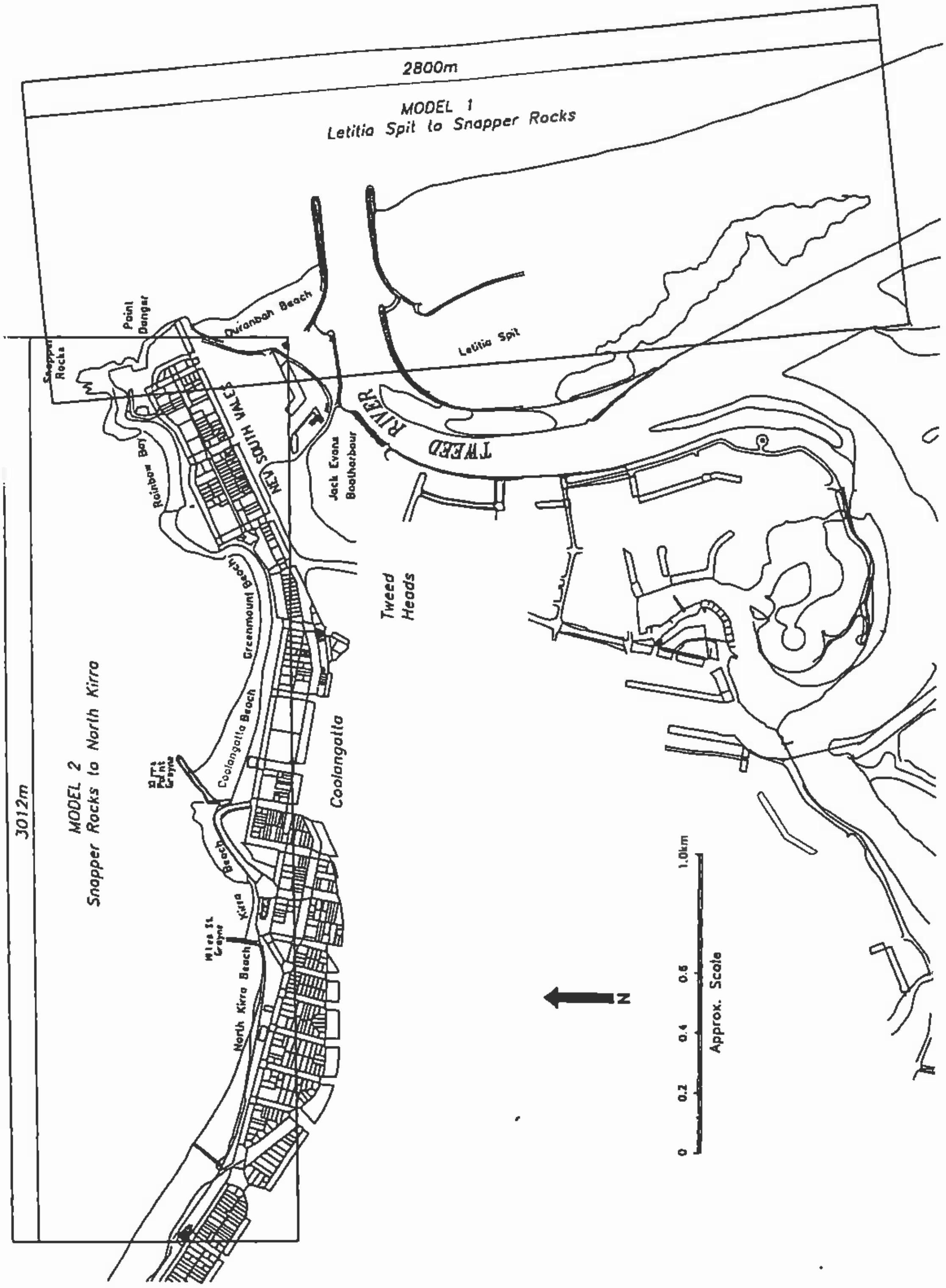


Figure 6.1
GENESIS Model Locations

- increased beach widths and recreational amenity; and
- significantly enhanced surfing conditions.

At the same time, there will be removal of sand from the northern Letitia Spit and Tweed River entrance nearshore area. This will have effects on the beach system and bar processes there, and will significantly reduce the sand supply to Duranbah which will respond to this reduction by realigning and retreating to establish a new equilibrium with the sand supply rate.

The Deed of Agreement and operating performance criteria offer opportunity to place up to an average of 50,000 cubic metres of sand per year at Duranbah and 75,000 cubic metres at Kirra. Options are considered to maximise the benefit of these supplies.

More detailed discussion of these impacts is set out below.

6.3.3 Impacts at Specific Beaches

Letitia Spit

Extraction of sand from the beach and nearshore profile south from the existing river entrance will directly change the nearshore bathymetry and beach shape. This would occur with, for example, a fixed trestle-mounted pumping system equivalent to that at the Gold Coast Seaway or dredging by mobile plant in that area. In that case, the following impacts would result::

(i) There would be a localised retreat of the beach itself and development of a new equilibrium beach shape.

(ii) Associated with the above, the dune system would become modified including some erosion until new equilibrium is achieved.

The beach response modelling undertaken using the software package GENESIS indicates the likely response of Letitia Spit to the bypassing. Such modelling provides a reasonable indication of likely behaviour but is subject to inherent model limitations and uncertainties in both the nature of the bypassing operation likely to be adopted and the response of the nearshore bathymetry to the sand extraction procedure. The nature and extent of shoreline retreat will depend on the particular bypass system adopted and its operational design.

Specifically, should the sand extraction occur in the form of regular dredging of the river entrance channel, as was undertaken in the Stage 1A dredging, then there will be little associated impact on Letitia Spit, consistent with the present situation. Should the zone of sand extraction extend south of the river entrance, and should it occur at a localised nearshore area (eg. a trestle system as at the Gold Coast Seaway), then the impact on Letitia Spit would tend to be more extensive at and near the extraction location.

The GENESIS modelling has provided for two sand extraction scenarios to augment the knowledge gained in monitoring the effects of the Stage 1A works. These are:

- localised extraction along the approximate alignment of the southern training wall, and
- localised extraction (eg. trestle system) at a location about 250 metres south of the training wall.

The results are illustrated in Figure 6.3. In the former case, the maximum impact on the Letitia Spit shoreline is immediately adjacent to the training wall, where a retreat of 30-40 metres is predicted. In the latter case, a maximum shoreline retreat of about 90 metres is predicted at the extraction location. Retreat

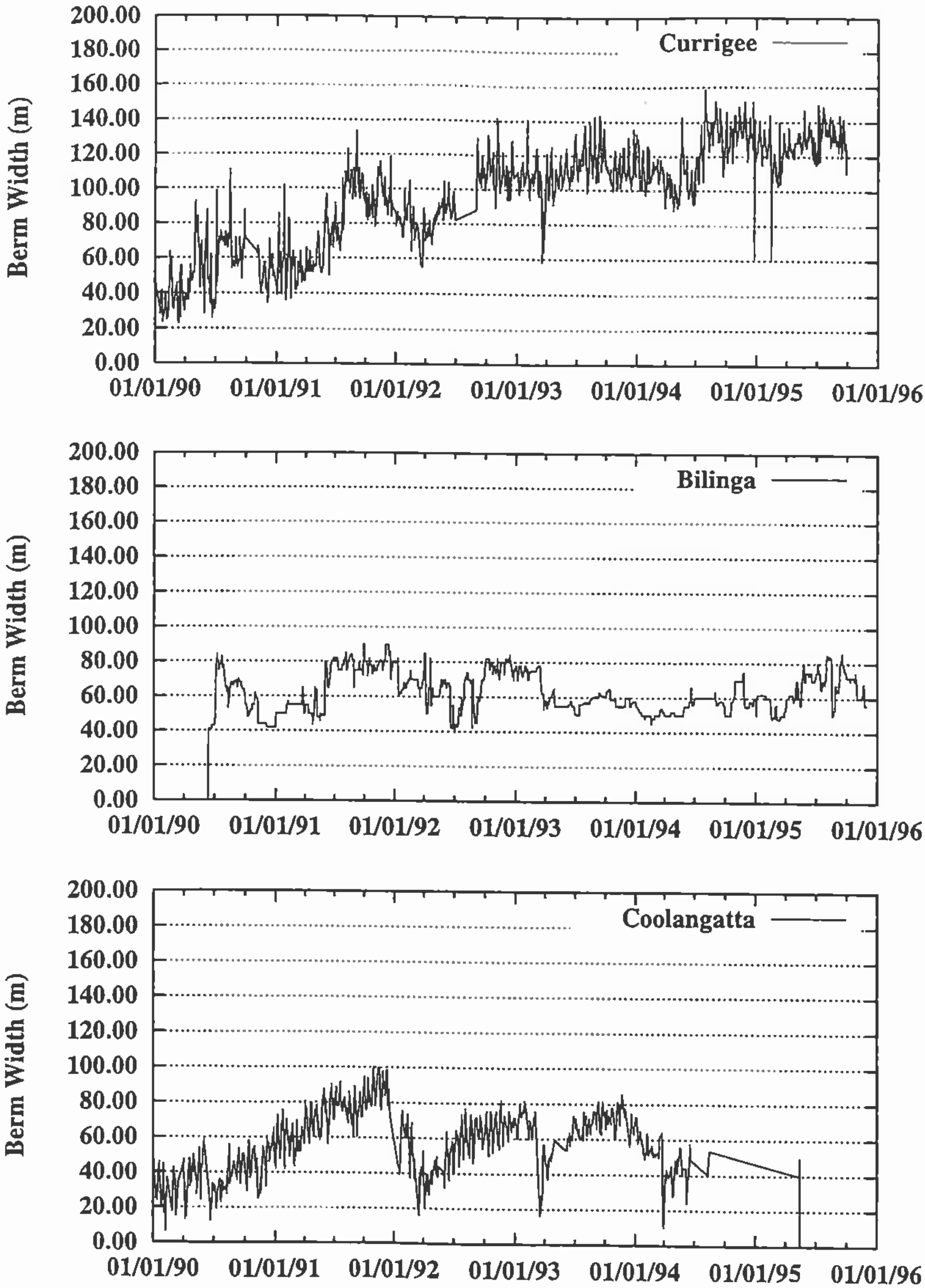


Figure 6.2
Natural Variability of Beach Widths From Daily COPE Measurements

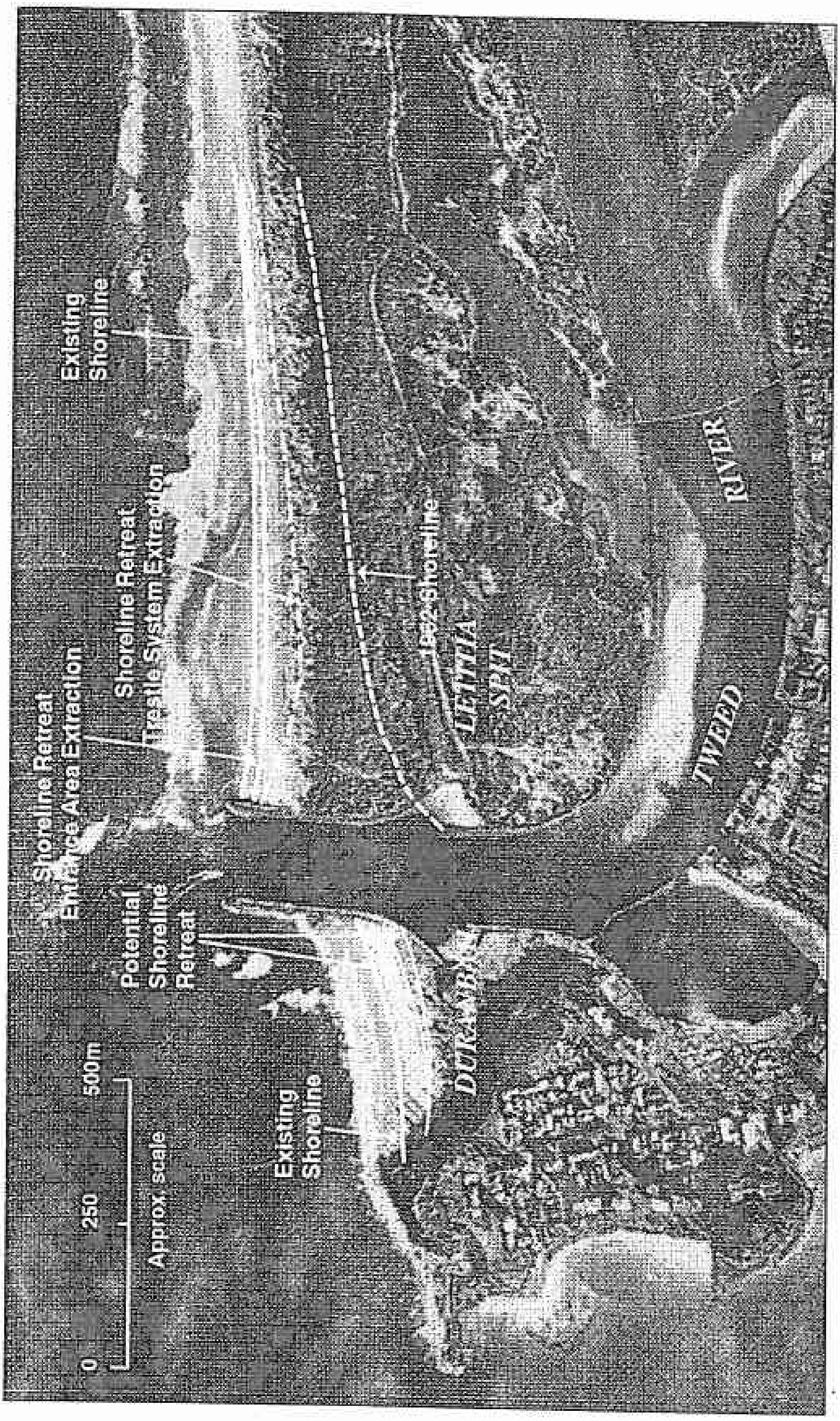


Figure 6.3
Bypassing Impact on Letitia Spit and Duranbah

would occur along about 1000-1500 metres of the northern end of Letitia Spit, decreasing in extent with distance away from the extraction location.

Clearly, for each scenario, normal beach fluctuations due to cross-shore transport associated with variations in wave conditions and in longshore supply of sand would continue, as indicated in Figure 6.2.

Accordingly, immediate localised impacts on Letitia Spit may be quite minor and of little consequence in terms of beach and dune impacts, or may involve shoreline retreat by up to about 90 metres, depending on which bypassing option is adopted. There may be other processes, such as long term slow leakages of sand to the Tweed River or other areas, which may cause slow long term retreat of parts of Letitia Spit. Such changes are likely to be imperceptible in the context of the short term beach variability.

In all of the above scenarios, the beach/dune system would adapt to a new dynamic equilibrium situation such that there will be no loss of beach width or capacity to accommodate storm erosion.

The bypassing project will have no effect on the shoreline or beach processes at southern Letitia Spit in the vicinity of Fingal. The beach and dune there will continue to fluctuate naturally as at present in response to the changing wave conditions and storms.

Duranbah

The existing sand transport pattern at Duranbah will be altered significantly by the bypassing project. The shallower river entrance bar system located about 300-400 metres offshore from the tip of the breakwaters will be depleted, reducing or preventing sand transport northward in that area.

The sand supply to the profile offshore from Duranbah will be reduced. Thus the profile there will tend to be depleted. Longshore sand transport will increasingly shift to the zone nearer the beach.

The particular response of Duranbah Beach to the sand bypassing will depend on a range of factors which, at this stage, cannot be predicted. These will be determined by the type and operational procedures of the bypassing system adopted.

The beach response will be determined by two principal factors, namely:

- reduction of the longshore sand supply, and
- changes to incident wave directions associated with changes to refraction patterns over the entrance bar and nearshore shoals.

With respect to the former of these, the beach condition soon after training wall construction provides a good indicator of the equilibrium beach position and alignment. Aerial photography (Figure 6.4) and particularly that taken in 1965 has been used, together with the GENESIS modelling to quantify the likely shoreline response. These indicate a shoreline retreat of about 50 metres from the present position, more or less uniform along the beach (Figure 6.5).

Changes to wave patterns cannot be predicted, as they depend intimately on the bypassing system, operational procedures and the extent of natural leakage of sand across the entrance channel to Duranbah. In the event of high leakage, the bar and shoals will continue to exist, albeit in reduced size and form, and the 1965 situation is probably a good indicator of the future beach alignment.

In the event that there is little sand leakage to Duranbah and the Duranbah Discharge Quantity (50,000m^3/year) is the only supply, then it is likely that the entrance bar and nearshore shoals will be largely removed. In that case, the waves will approach the beach more from the southeast and the beach alignment will tend to rotate clockwise. There could thus be greater shoreline retreat near the southern end, estimated to be up to about 80-90 metres from the existing position in the worst case scenario.

Rainbow Bay

The shape and condition of the beach at Rainbow Bay is largely dependent on the condition of the offshore shoals around Snapper Rocks. With the bypassing plant operational, it is likely that the offshore shoals around Snapper and Marley Rocks will be consistently full, ensuring a strong persistent sand supply to this area.

This will represent a return to the natural condition existing prior to training wall extension, characterised by:

- strong and persistent development of the nearshore shoal extending directly past Rainbow Beach from Marley Rocks to Greenmount Hill;
- strong and persistent longshore sand transport along this shoal to Greenmount;
- the common occurrence of a nearshore lagoon between the shoal and the main public beach, in which wave action and currents are relatively calm;
- increased occurrence of a wider recreational beach;
- separation of areas of general beach and surf use from areas used by surfboard riders; and
- increased nearshore sand buffer against excessive beach erosion during storm events;

Despite this, the beach will behave dynamically, with beach width and nearshore shoal bathymetry varying considerably over the short to medium term associated with natural variability of wave conditions and storm erosion events.

Greenmount/Coolangatta

The condition of the beach at Greenmount is largely dependent on the flow of sand around Greenmount Hill and the transport away from the beach past Kirra Point. The supply of sand to this area will be improved as a consequence of the improved supply to and past Rainbow Bay and the effects of the Stage 1 dredging in restoring the nearshore profile bathymetry.

As at Rainbow Bay, the behaviour of Greenmount Beach will return to that which characterised the area prior to training wall extension. This includes persistent occurrence of a wide recreational beach and frequent strong sand supply in the form of a spit-like shoal extending past Greenmount Hill. This shoal may or may not be attached to the beach in the area towards Coolangatta, but typically will create a lagoon between it and the main recreational beach.

This spit/lagoon bathymetry is not represented properly in terms of shoreline position in the one-line schematisation of GENESIS. Instead, an equivalent volume-related shoreline response is indicated, as shown in Figure 6.6, which suggests greater beach width benefit than would actually occur since much of the sand will remain in the nearshore shoals. The shoal bathymetry will be highly variable in response to varying wave conditions. It will provide an increased sand buffer which will help to minimise storm erosion of the recreational beach. The beach width will vary significantly in the short to medium term, and storm erosion will continue to occur from time to time.

Similarly, Coolangatta will receive a more persistent littoral sand supply which will move past Kirra Point under the natural wave/current action. The Kirra Groyne creates a different beach and sand transport pattern from that existing prior to the training wall extensions. It has the effect of stabilising Coolangatta

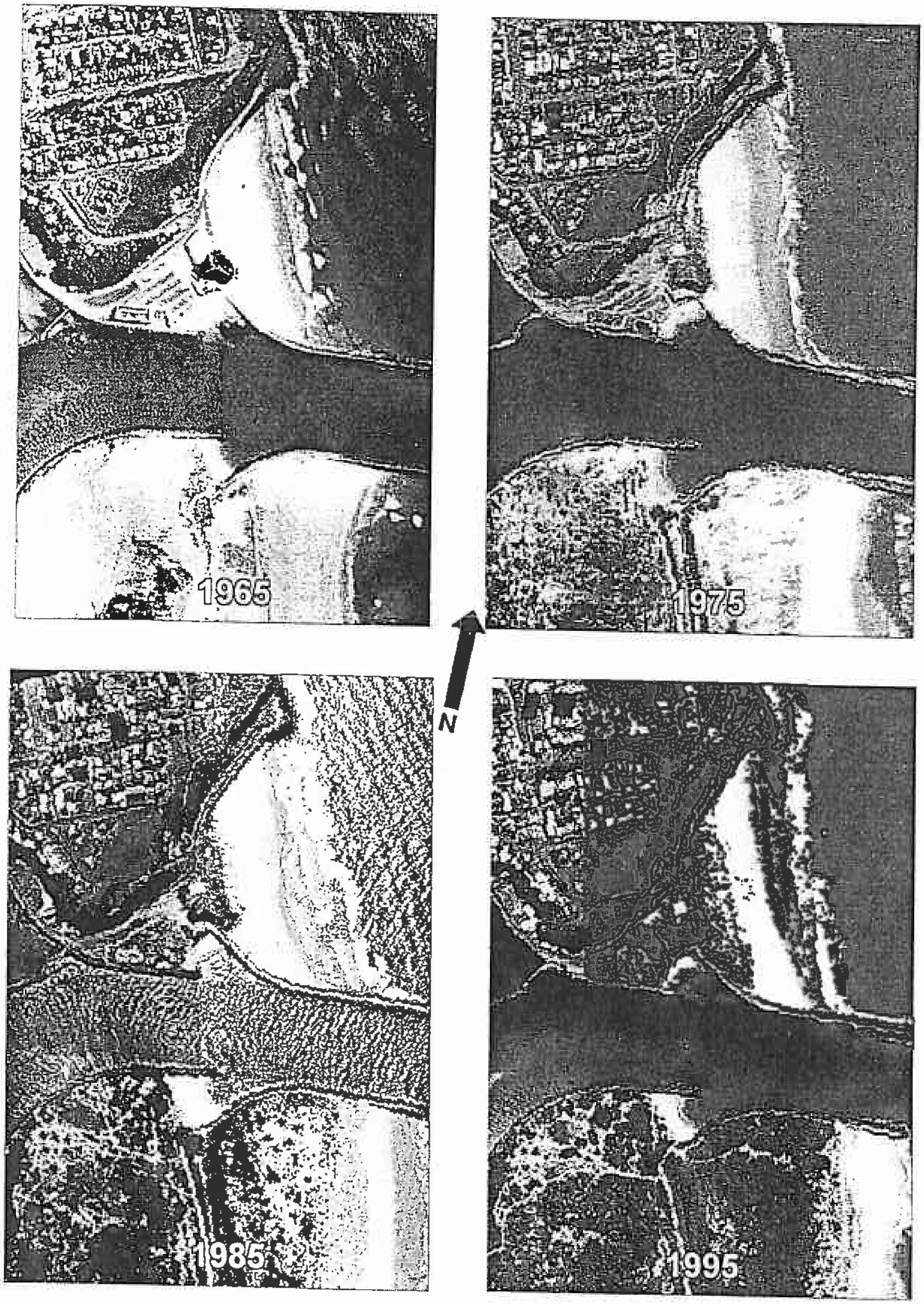


Figure 6.4
Historical Aerial Photographs - Duranbah

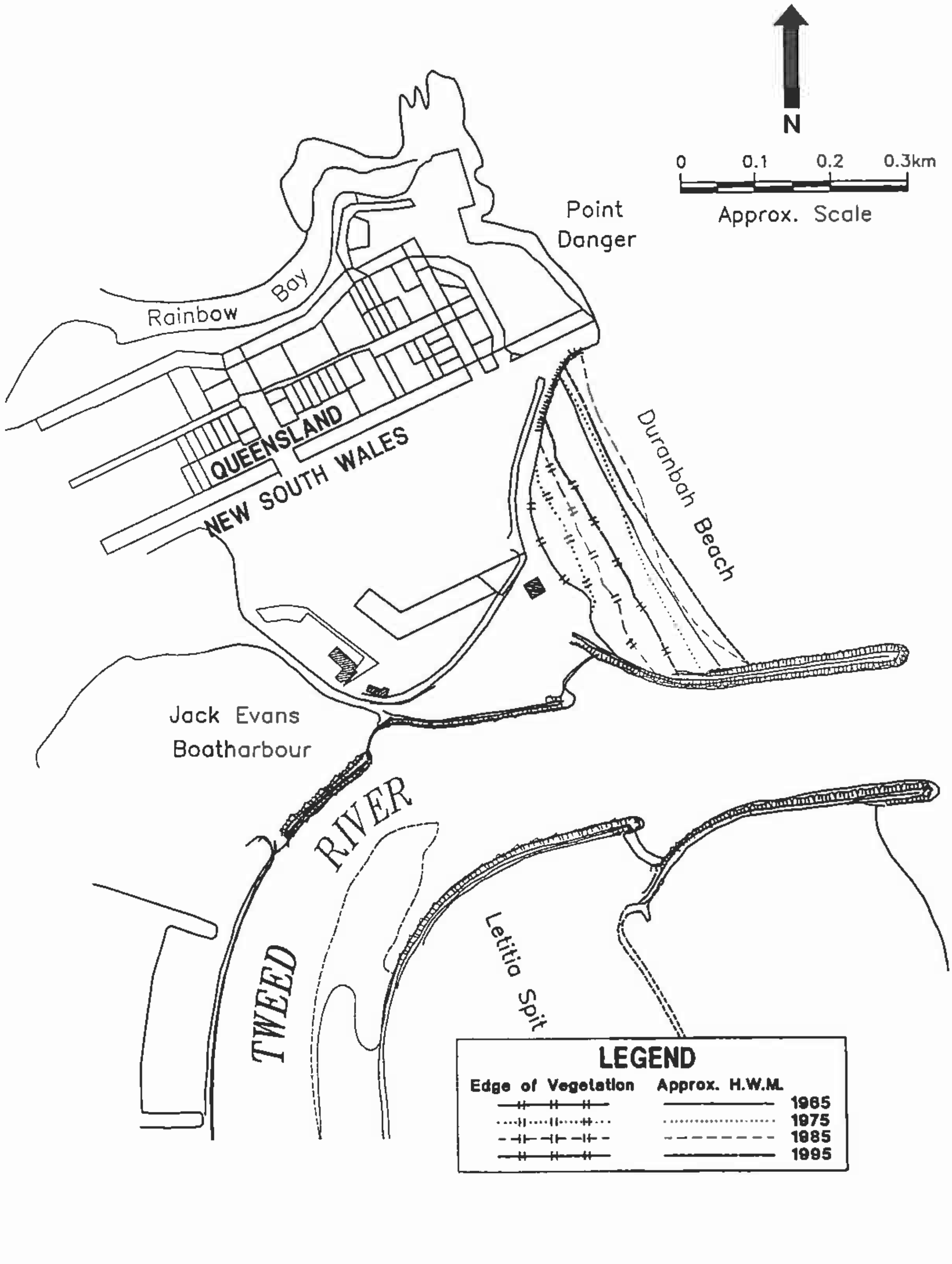


Figure 6.5
Historical Shorelines at Duranbah

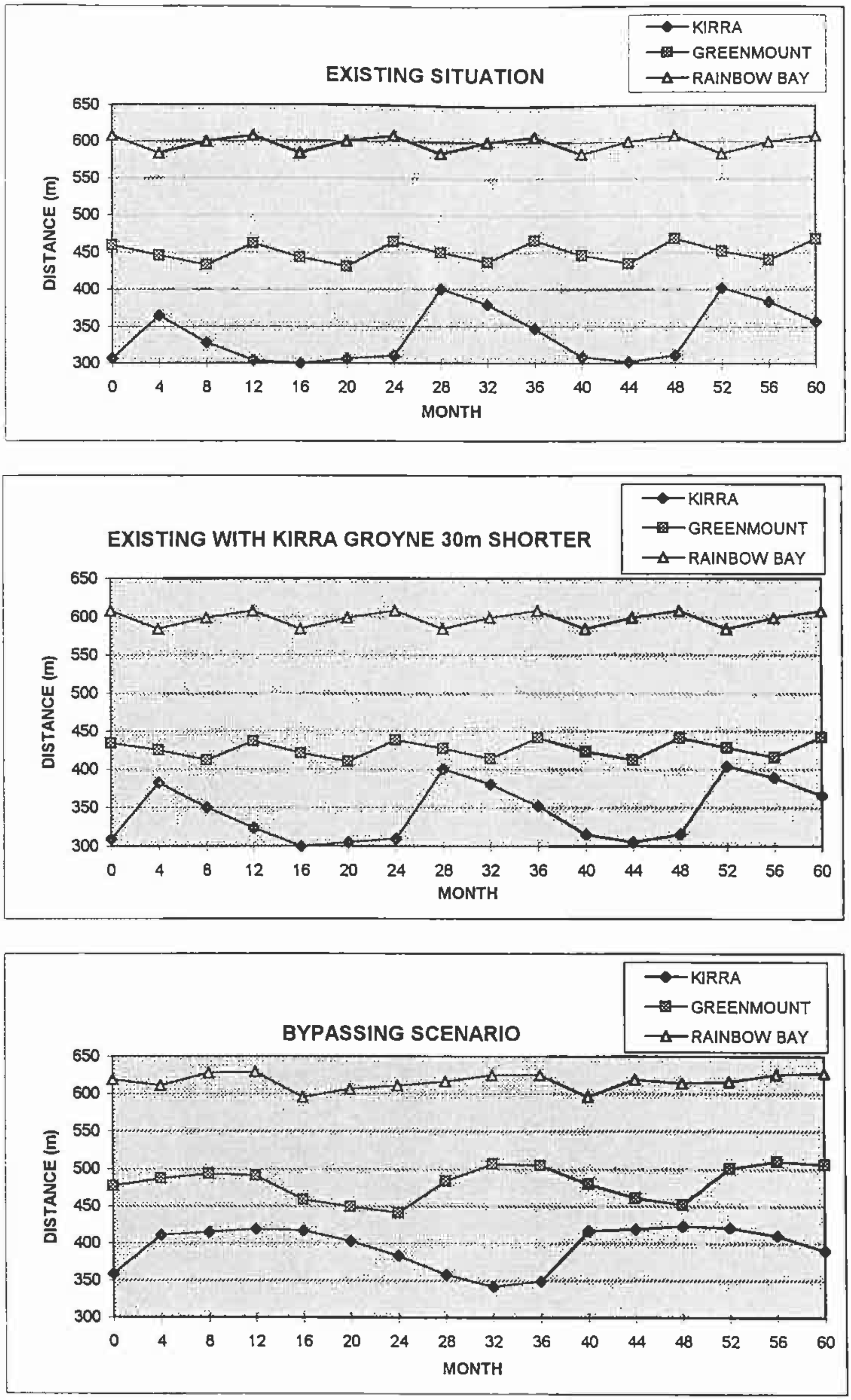


Figure 6.6
Bypassing Impact on Southern Gold Coast Beach Widths

Beach with greater width than before, and may alter the beach/bar alignment in such a way that the sand supply past Kirra Point is somewhat more variable than was originally the case.

Kirra

The condition of the beach at Kirra is dependent on the nearshore profile bathymetry, the associated wave refraction patterns, and the transport of sand past Kirra Point. The Kirra Point and Miles Street groynes influence the sand transport patterns and beach conditions.

The Stage 1A dredging has not yet restored the natural nearshore profile bathymetry in this area. Sand deposited in the deeper water and outside the Kirra Reef Exclusion Zone has not yet been redistributed by the waves and currents to the longer term equilibrium condition. Such condition is characterised by greater sand volume in the area out to about the 6 metre contour than present exists and correspondingly less in the sand deposition area further offshore.

Kirra Beach and the beaches further north are fed by sand moving along the shoreline and sand moving through the reef area in longshore and cross-shore directions. Essentially, the area in the vicinity of the reefs is a rocky substrate of which most is covered by sand, with that part not covered by and constituting the reef. Historically, Kirra and Kirra Central beaches have fluctuated, about their mean position and, for Kirra Central the landward extent of fluctuation, has been seaward of its recent position.
The beneficial impact of the bypassing project at Kirra will be restoration of the full littoral sand supply and associated increase in beach width. This is of particular importance for Kirra (Central) which is located at the downdrift end of the series of natural headlands and artificial groyne structures. These controlling features help stabilise the beaches immediately updrift and, in so doing, tend to focus the erosional effect of any sand supply deficit at Kirra.

The results of the GENESIS modelling for Kirra (immediately north of the Miles Street groyne) are shown in Figure 6.6. As at Greenmount, the apparent beach width benefit indicated by the model overestimates the probable actual improvement to be achieved. Nevertheless, the modelling shows that the beach width will be increased substantially and should tend to return to conditions existing prior to 1962 over time as the sand supply feeds through the system and the nearshore profile in the vicinity of Kirra Reef readjust to the natural equilibrium situation. An interesting feature of the model result for Kirra is the indicated change in the periodicity of beach width fluctuation with increased sand supply, exhibiting 3 year cycles for the input one year cycles of wave data.

Despite restoration of the long term average sand supply to the required 500,000m^3/year along all these beaches, Kirra beach will continue to be subject to sand supply variability caused by varying wave conditions and the updrift headland and groyne features. Resulting beach width variability can be moderated within the Deed of Agreement provisions by direct placement of (average) 75,000m^3/year of sand as required.

Thus, it is considered most likely that, over time, Kirra Beach and Kirra Central will be restored to conditions with the general shoreline position seaward of that presently existing, similar to that existing prior to training wall extension (refer Figure 6.7), subject to the additional influences of:

- the Kirra Point and Miles Street groynes, which will tend to create increased beach variability, and
- direct placement of sand from time to time, which will tend to moderate beach variability.

Coolangatta Creek/North Kirra

The beach at Coolangatta Creek and past North Kirra is presently relatively wide as a result of the past beach replenishment works. The beach and nearshore system in this area is continuing to adjust slowly to those works under the influence of the prevailing wave, current an wind conditions.

The precise nature of the final equilibrium state of this beach area is uncertain, but is likely to eventually replicate that of the early 1960's (refer Figure 6.7). Thus, continuing evolvement of the beach there is likely to include:

- little change In the location of high water mark from the present location at and north from Coolangatta Creek, and
- development of a wind formed dune system at the back of the beach along Kirra Central to North Kirra.

Management of the sand supply past Snapper Rocks to moderate sand transport pulsing combined with a supplementary outlet at Kirra will ensure a reduction in the variability of Kirra beach. It is also probable that shortening or removing the groyne at Kirra Point will reduce the variability both the sand supply past the headland and the width of Kirra beach.

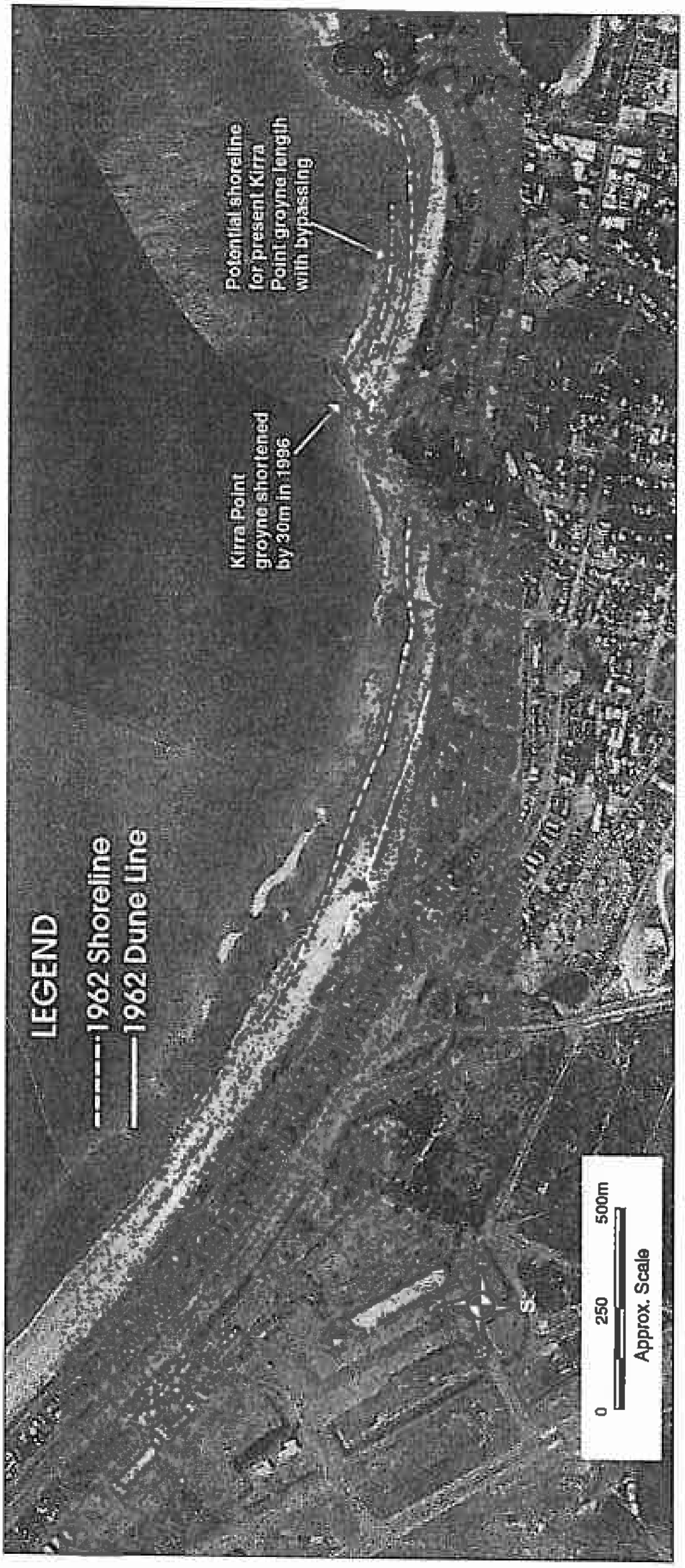


Photo: 25 May 1994

Figure 6.7
Historical and Projected Shorelines - Greenmount and Kirra

7 Tweed River Entrance

7.1 General Considerations

Linkage of the Tweed River ESTRY tide model to the coastal zone TUFLOW model allowed modelling of the complex patterns of currents and sand transport in the vicinity of the river mouth. Interaction of the tidal flow to and from the river with the longshore currents generated by wave, wind and EAC forcing were thus able to be modelled.

To facilitate this modelling, the 2-Dimensional coastal models extended some 400 metres upstream in the river entrance, where the link to ESTRY was effected. Representation of the river training walls was rather coarse, given the grid element finite difference schematisation, and involved orienting their alignment east-west rather than 7º north of east as in the prototype.

It must be noted also that this modelling cannot represent all of the physical processes taking place, particularly those associated with depth varying current activity. Thus, the modelling results for the river mouth area are regarded as indicative of the gross processes only.

7.2 River Mouth Currents

Modelling of the river mouth area has been undertaken for three entrance bathymetry configurations, namely:

- pre-dredging (April 1995)
- entrance channel dredged (May 1996)
- potential bypassing scenario

The model bathymetries were based on surveyed contour charts for the first two cases. For the bypassing scenario, the model bathymetry assumes dredging to a depth of -6.5mAHD over an extensive area seaward of the training walls.

In each case, the tidal flow to and from the river has been combined with typical modal swell and wind conditions. Thus the effect on current patterns of changing the river mouth bathymetry is identified.

The results of this modelling for both the peak flood and peak ebb tide conditions are presented in Figures 7.1 (a)-(c). They indicate:

- the flood tide currents tend to enter the river more or less radially, with a slight offset biasing the inflow to favour the southern side due to wind and wave influences;
- the ebb jet remains quite well defined for a distance of some 400-600 metres offshore from the training walls, with the development of relatively large adjacent eddies which tend to be transient and/or variable depending on the influence of longshore wind/wave forcing;
- the dredging of the entrance to May 1996 had only slight effect on the currents, with a slight increase on the velocities between the training walls and little effect nearshore where the ebb jet continued to be directed more or less directly seawards over the shallow bar with only slight northward deviation; and
- improvement of the bar associated with the bypassing will simplify the ebb jet pattern and reduce its intensity outside the training walls where the water depths are expected to be increased.

Additional modelling with representation of the EAC has also been undertaken. The results are presented in Figure 7.2 for the existing (1996 dredged) situation. Key features of these results are as follows:

- During the flooding tide, the EAC runs southward alongshore quite close to Point Danger and Duranbah and contributes to the radial inflow to the river entrance. The radial inflow from south of the entrance is partially negated. Any upcoast wind/wave current tends to negate or reverse the EAC closer to the shore.
- During the ebbing tide, the EAC is deflected further offshore by the ebb jet, reducing its nearshore influence at Point Danger and Duranbah. For the 1996 dredged case, the model shows a large-scale eddy on the southern side of the river, with an upcoast longshore current along the northern end of Letitia Spit, strongly reinforced by the upcoast wave induced current in the surfzone.

It is understood that recent observations and measurements of current in the region and consistent with the modelled current patterns.

7.3 Sand Transport Patterns

The sand transport patterns around the Tweed River entrance have been modelled as described in Section 4. The results for the existing (pre-dredging) scenario and a potential bypass scenario have been plotted in Figures 7.3(a)-(d) for typical northeast sea (2m), southeast swell (2m), east storm (4m) and southeast storm (4m) wave conditions respectively.

It can be seen that in the existing situation, relatively high sediment transport rates occur over the entrance bar. These rates reduce substantially for the dredged bypass scenario with the higher rates being confined to a narrower zone close to shore.

7.4 Entrance Channel Infilling and Bypassing

The dredged entrance channel will be subject to progressive inflow of sand as part of the natural process of longshore transport. A channel which forms a 'slot' within adjacent shallow areas will receive inflow from both the south and the north, being the gross longshore transport. Over time, as the northern (Duranbah) side becomes depleted, transport back to the entrance channel will reduce and the inflow to the channel will approach the net longshore transport.

The proportion of the inflowing sand which will remain as siltation in the entrance channel (the trapping efficiency) is the difference between that transported into, and that transported out from within the channel. This is a function predominantly of the relative depth of the channel and the inflowing transport rate. The situation is more complex for a wide channel, in which sand may be deposited near one side while substantial erosion occurs at the other side.

A computational approach has been adopted to assist in estimating the likely trapping efficiency of various depths of channel. This has involved a simplified approach based on the relatively narrow channel assumption in which the longshore current speed in the channel is calculated from continuity of that outside, being more or less in inverse proportion to the depths.
The computation has been based on the full range of prevailing deep water wave heights in the region. Despite that, no attempt has been made to incorporate comprehensive wave refraction or two-dimensional effects. The intent is to indicate the relative trapping efficiencies for various channel depths.

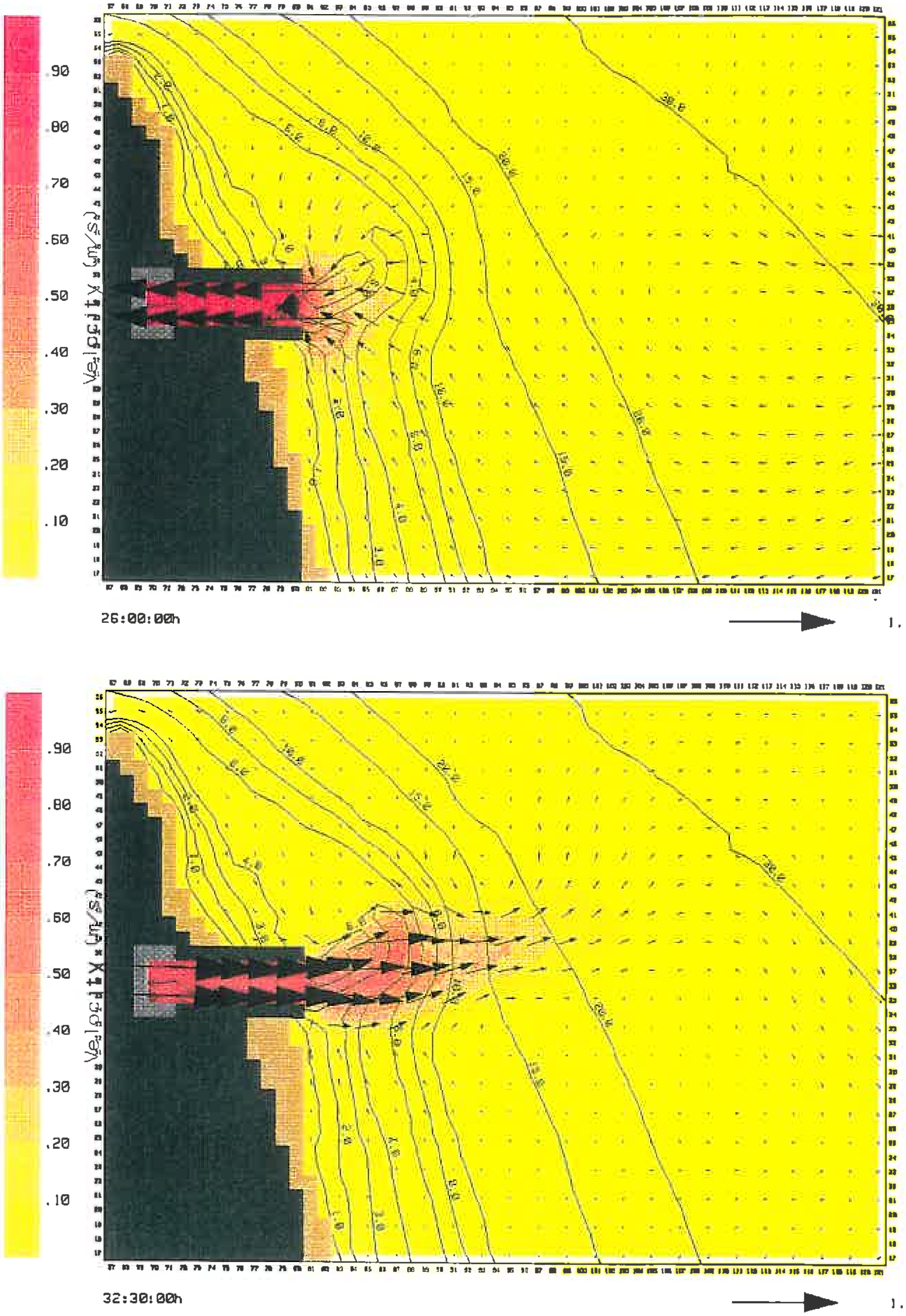
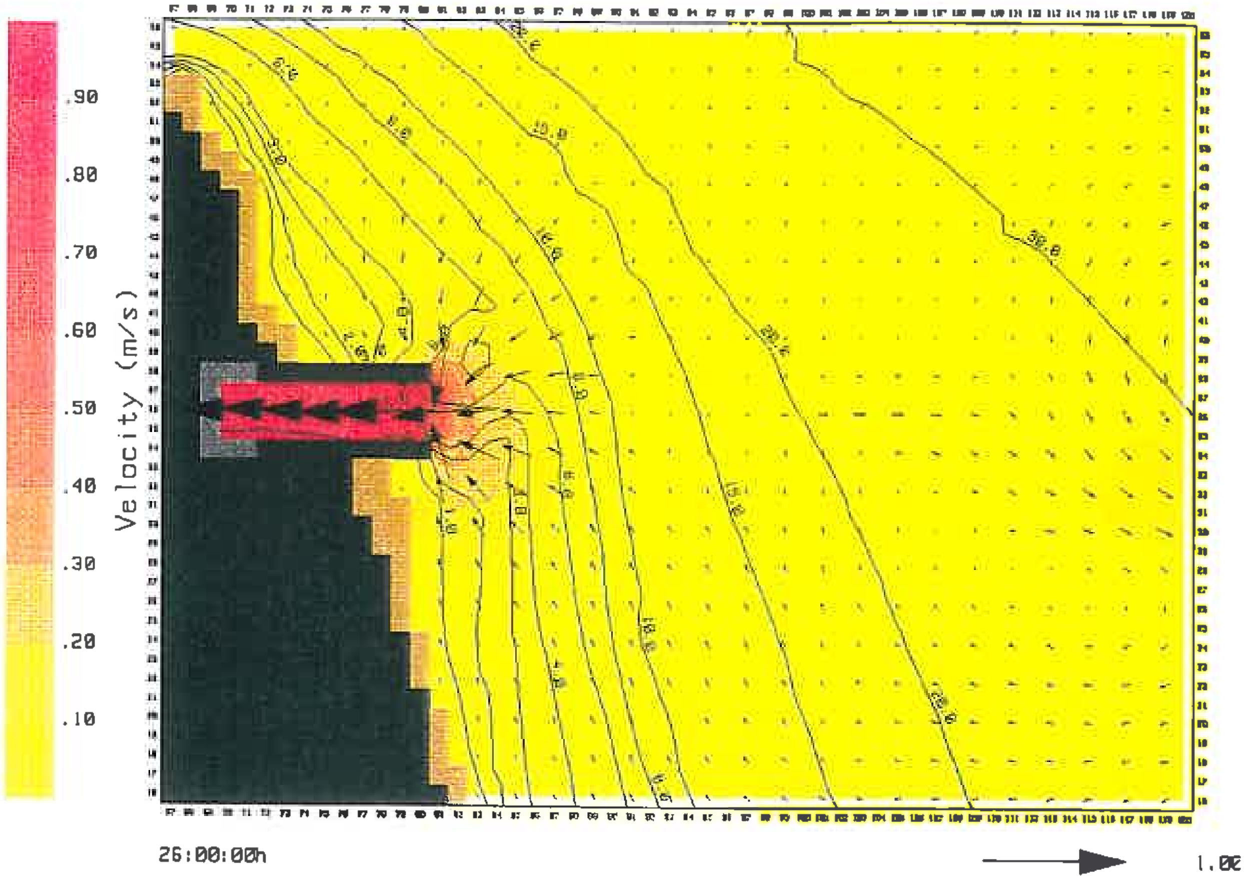


Figure 7.1a
Tidal Currents at Tweed River Entrance
Pre-Dredging Bathymetry



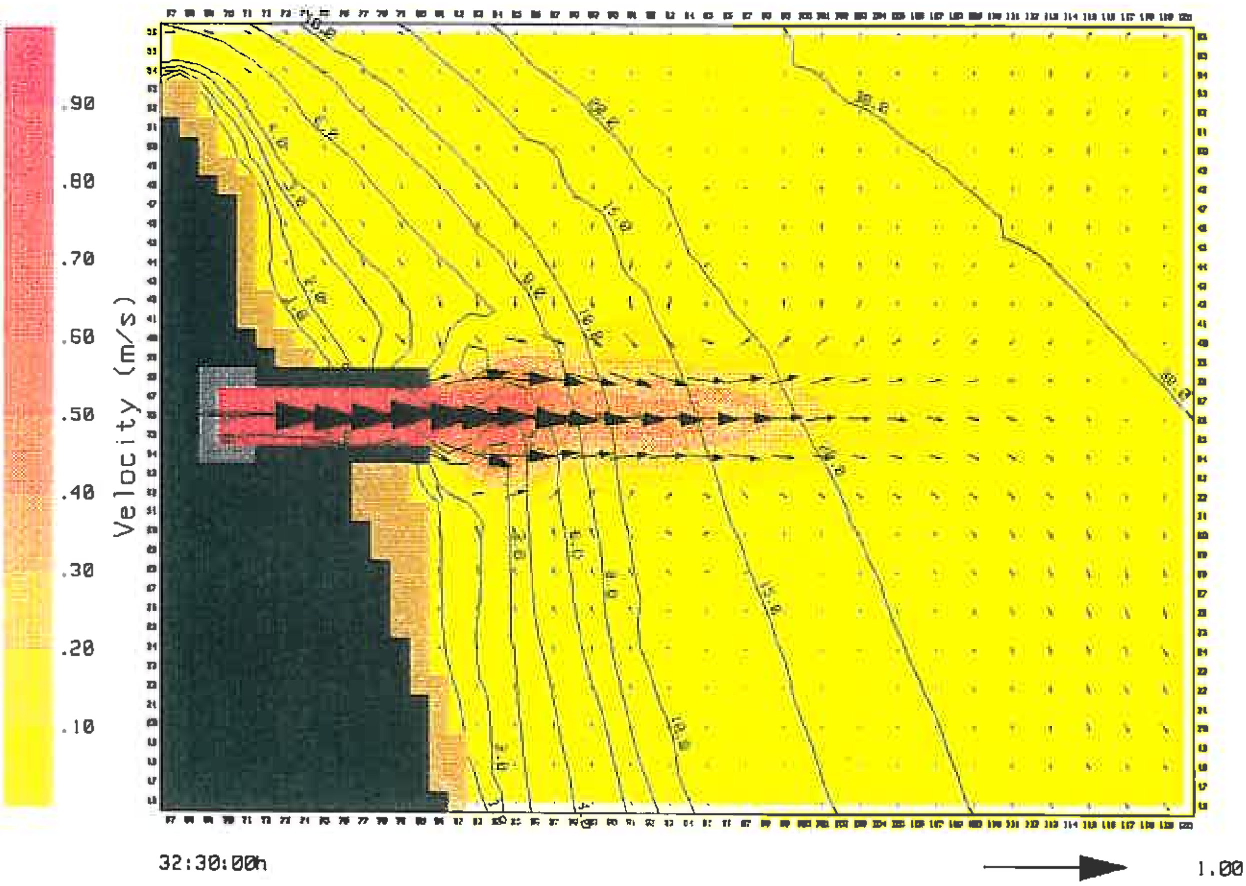


Figure 7.1b
Tidal Currents at Tweed River Entrance
Dredged (May 1996)

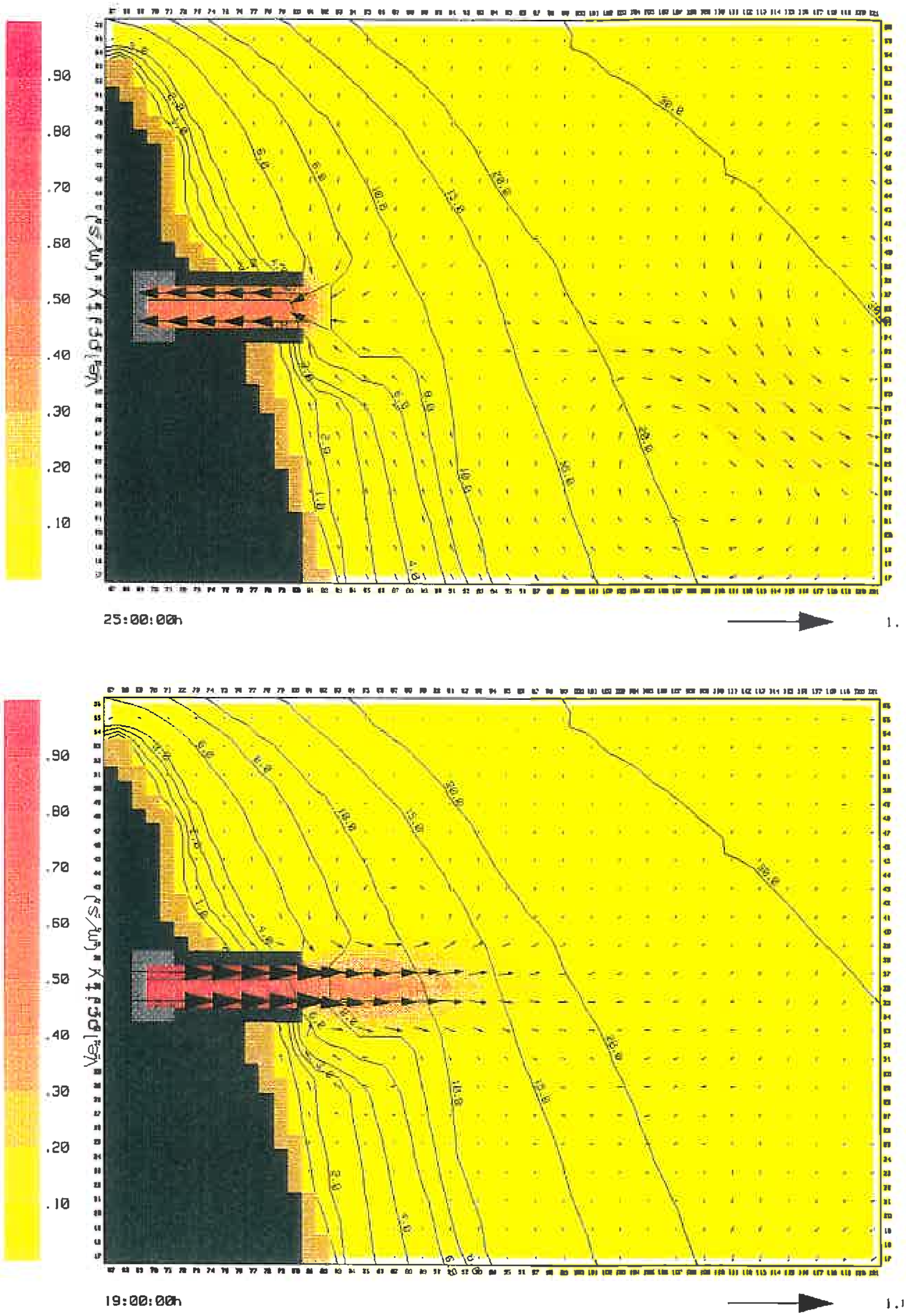


Figure 7.1c
Tidal Currents at Tweed River Entrance
Bypassing Scenario

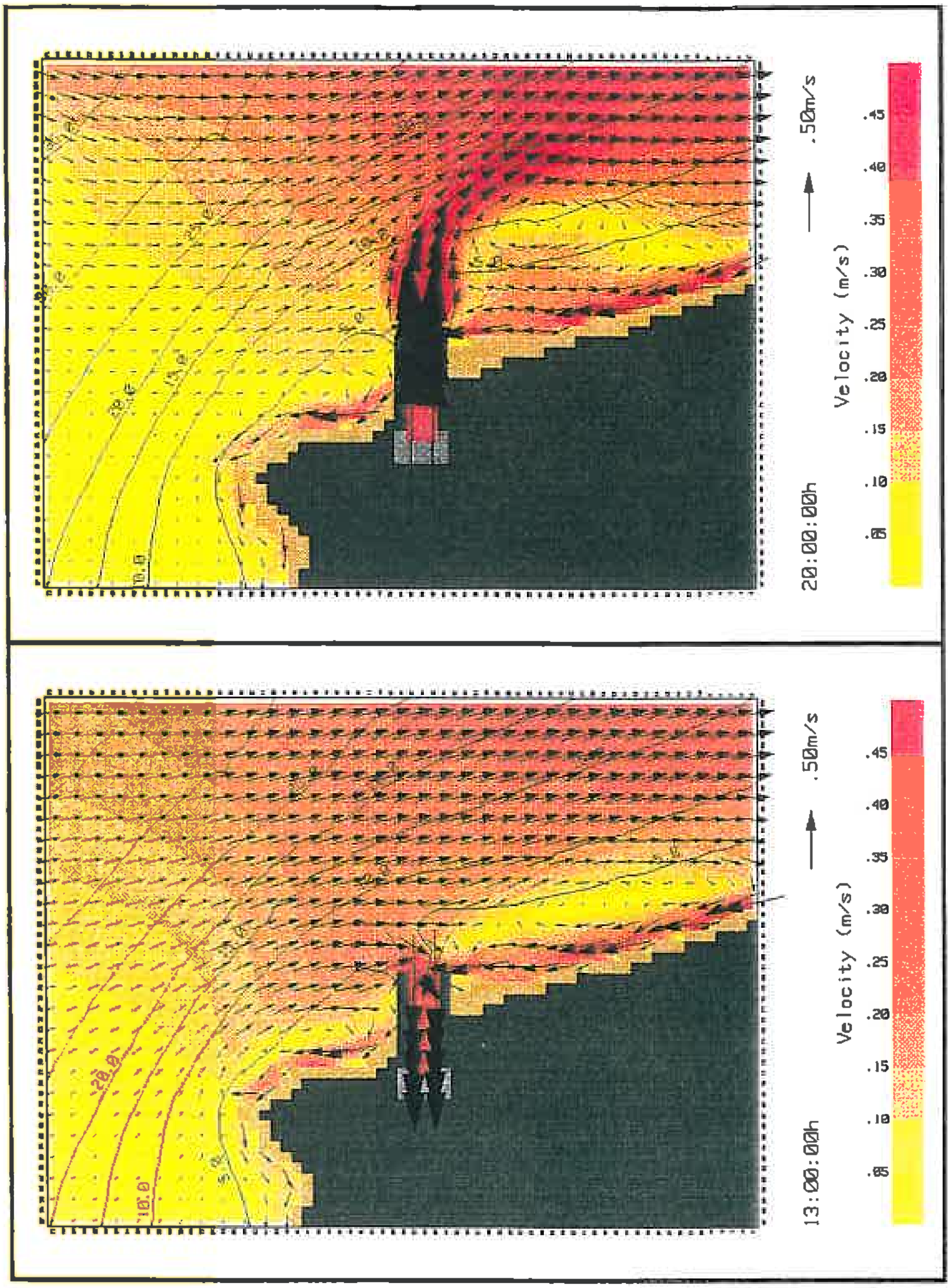
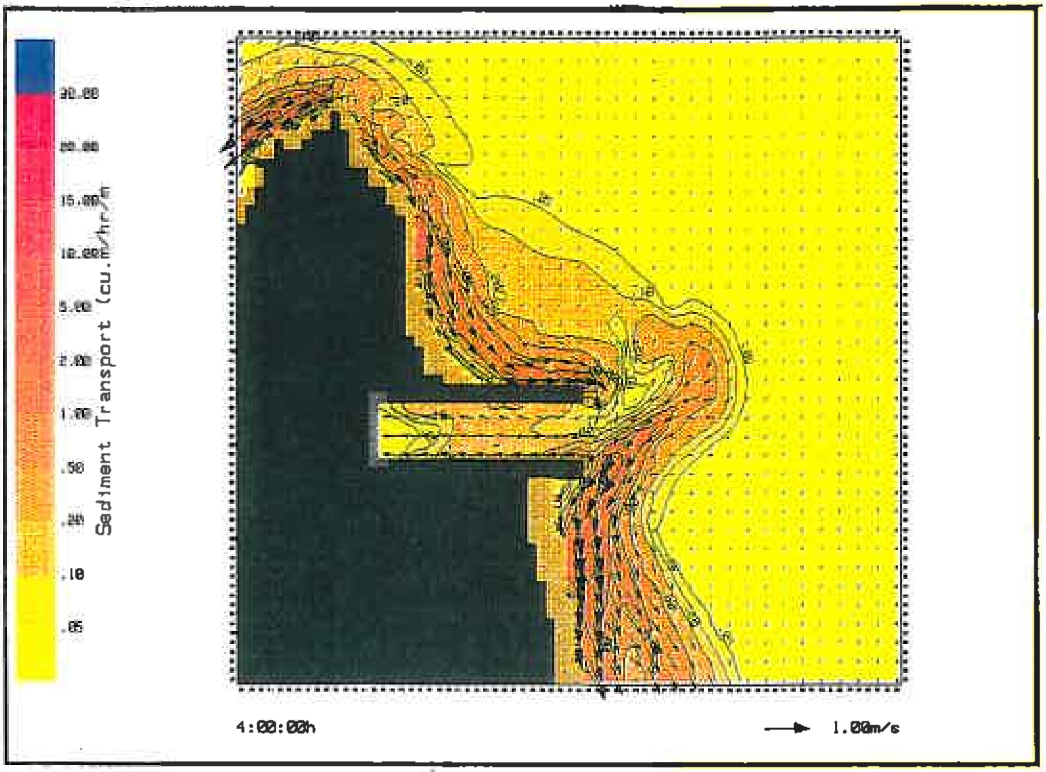
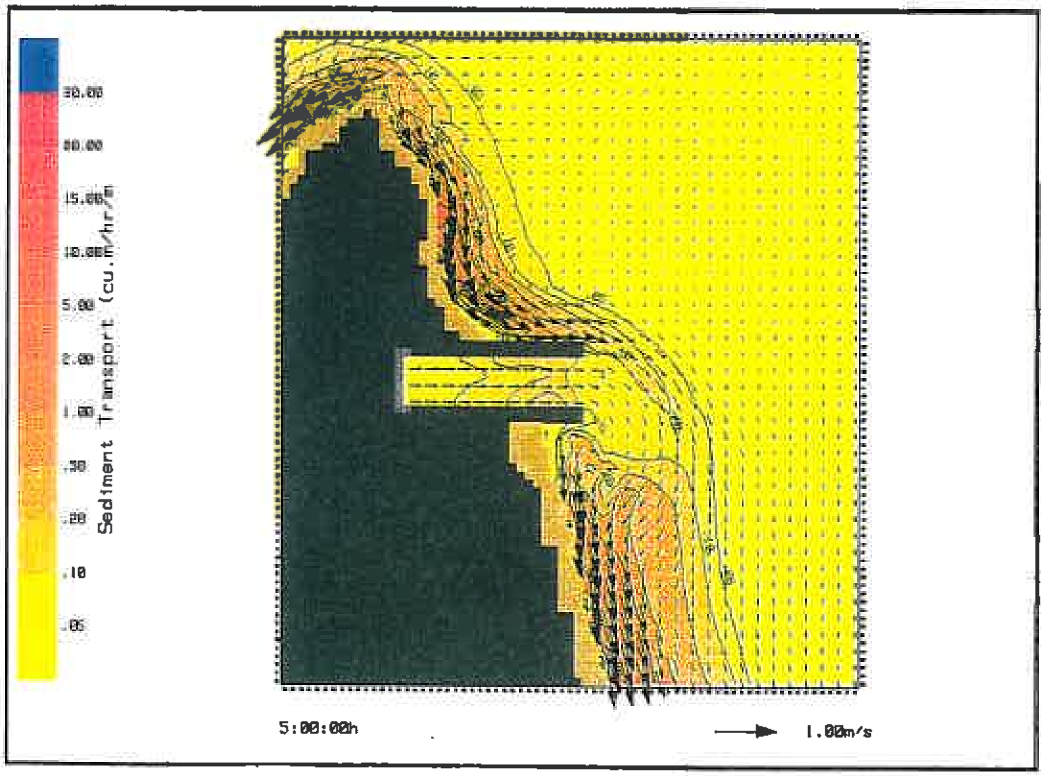


Figure 7.2
East Australian Current

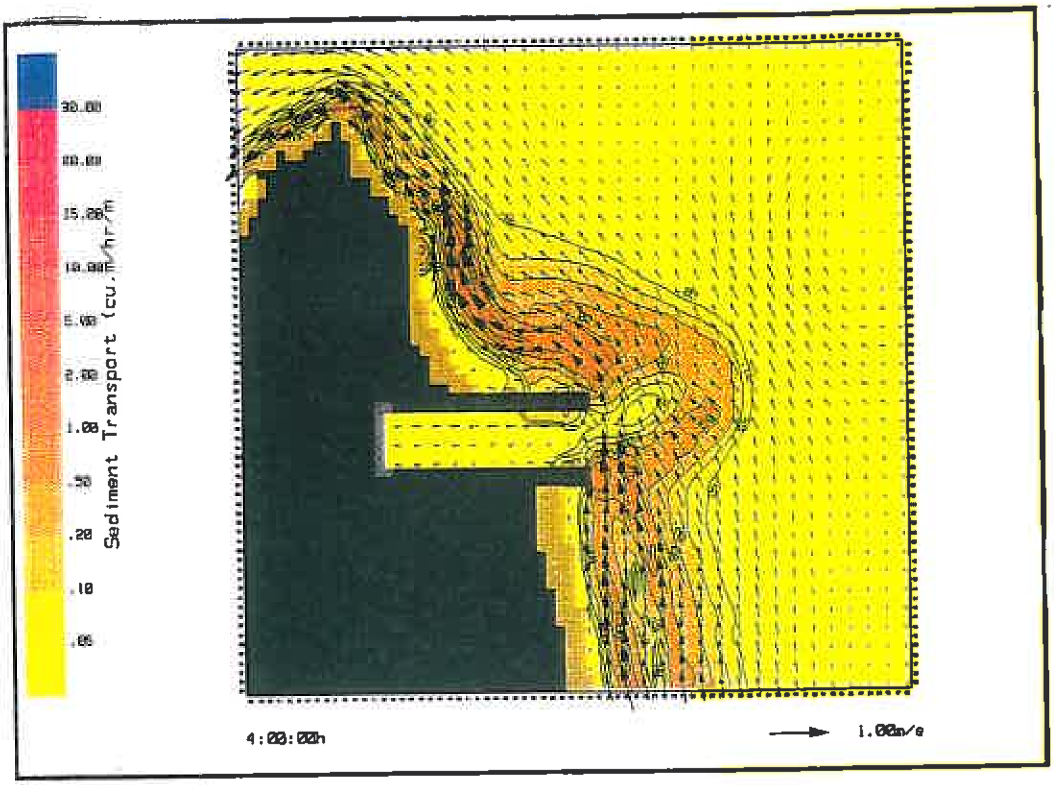


Northeast Sea (2m6s) - Existing

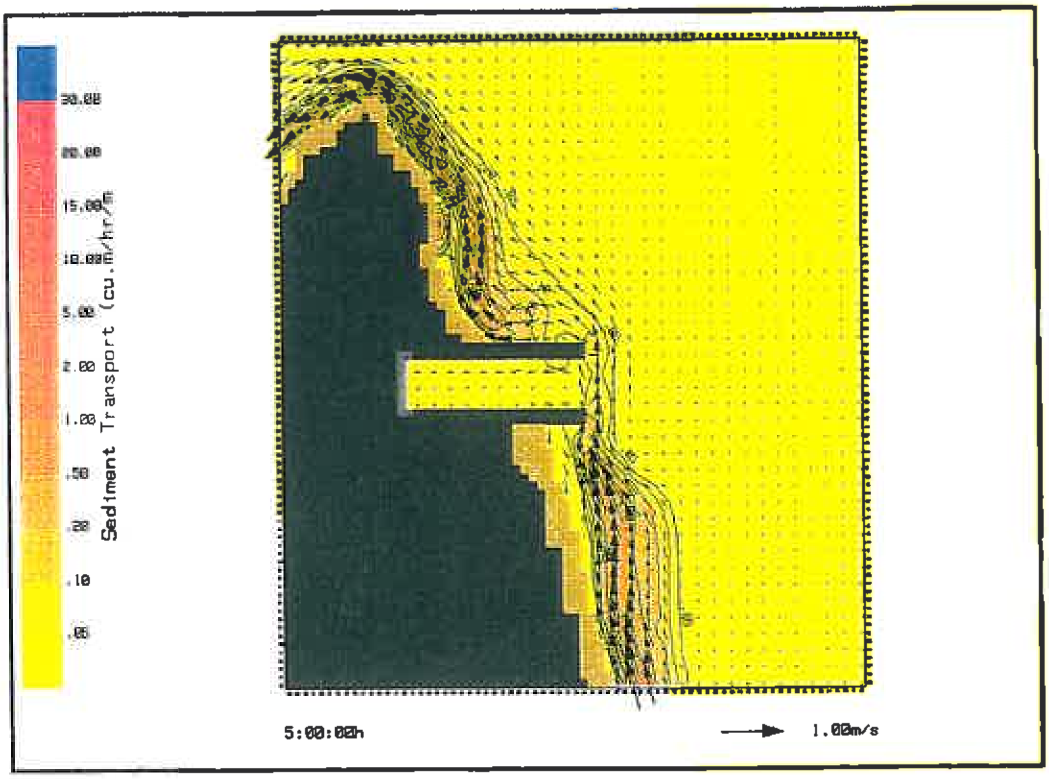


Northeast Sea (2m6s) - Bypassing

Figure 7.3a
Impacts on Sand Transport Patterns - Northeast Waves

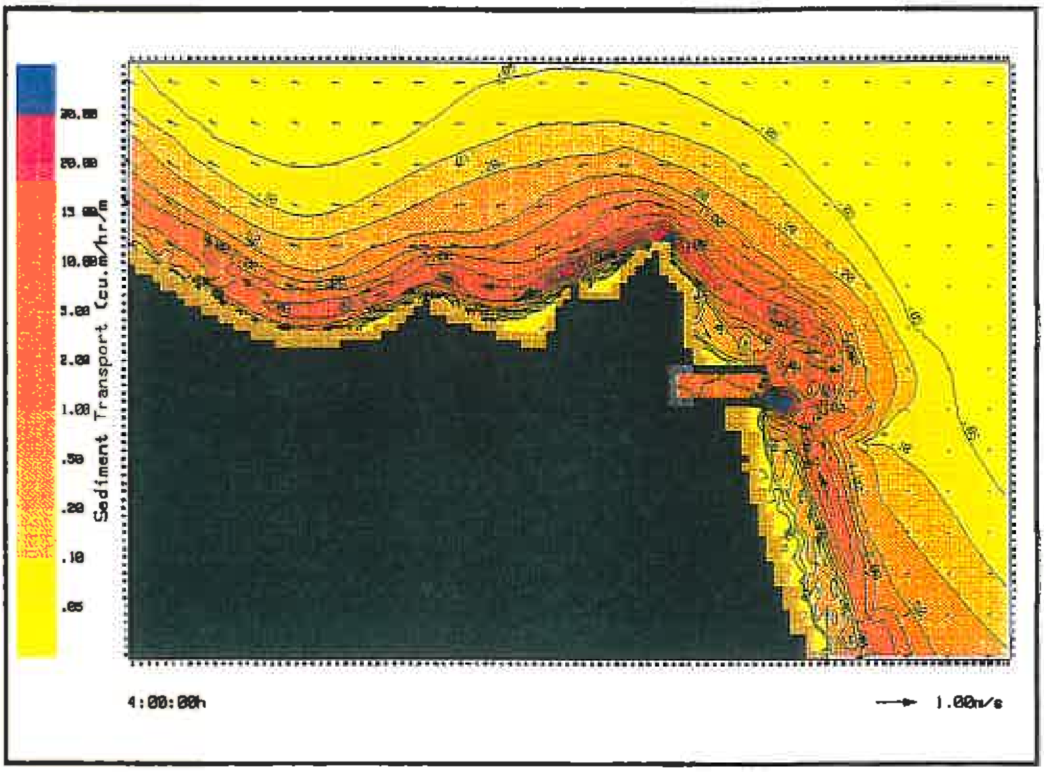


Southeast Swell (2m10s) - Existing

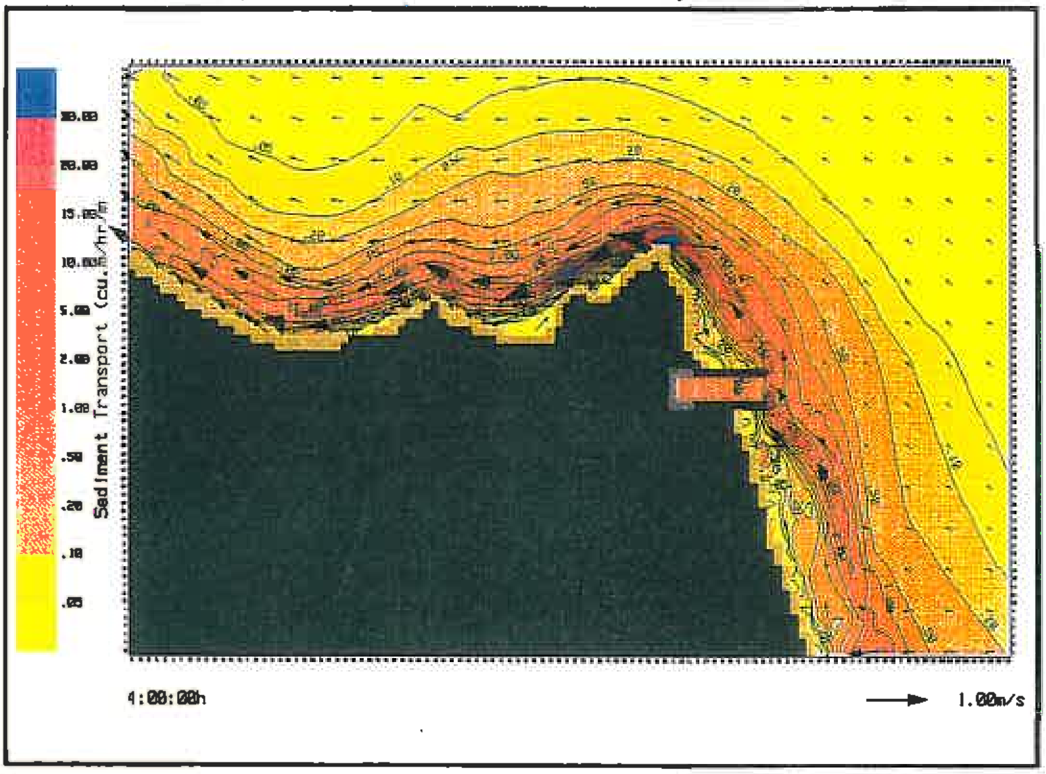


Southeast Swell (2m10s) - Bypassing

Figure 7.3b
Impacts on Sand Transport Patterns - Southeast Swell

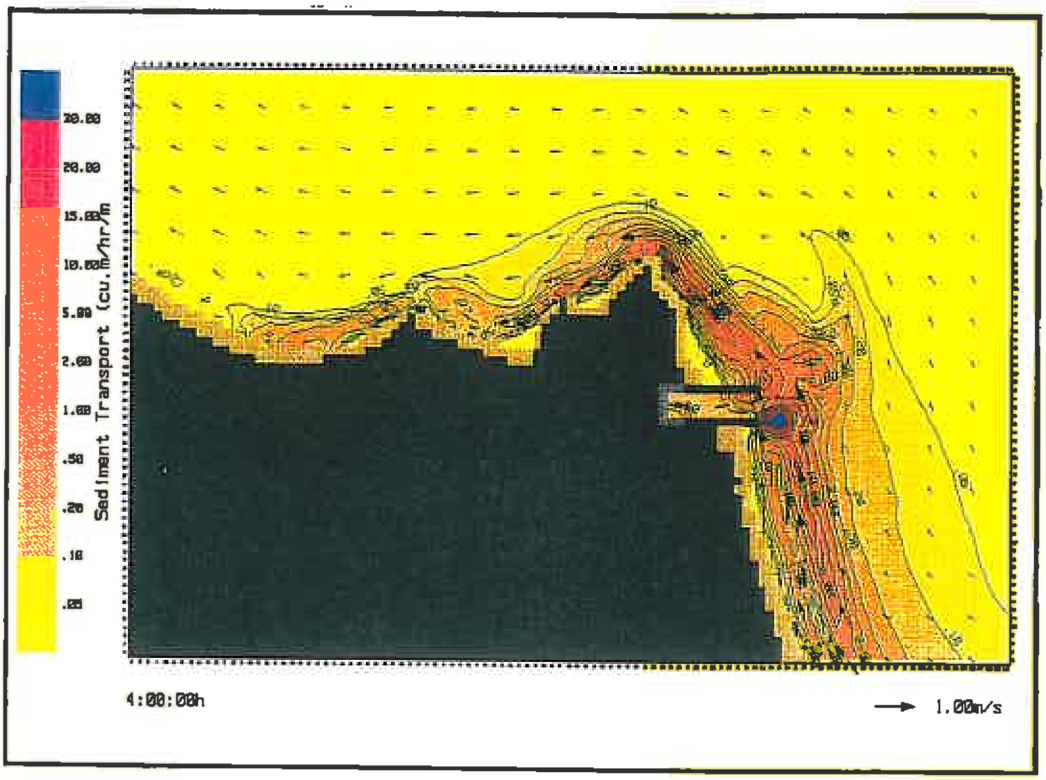


East Storm (4m9s) - Existing

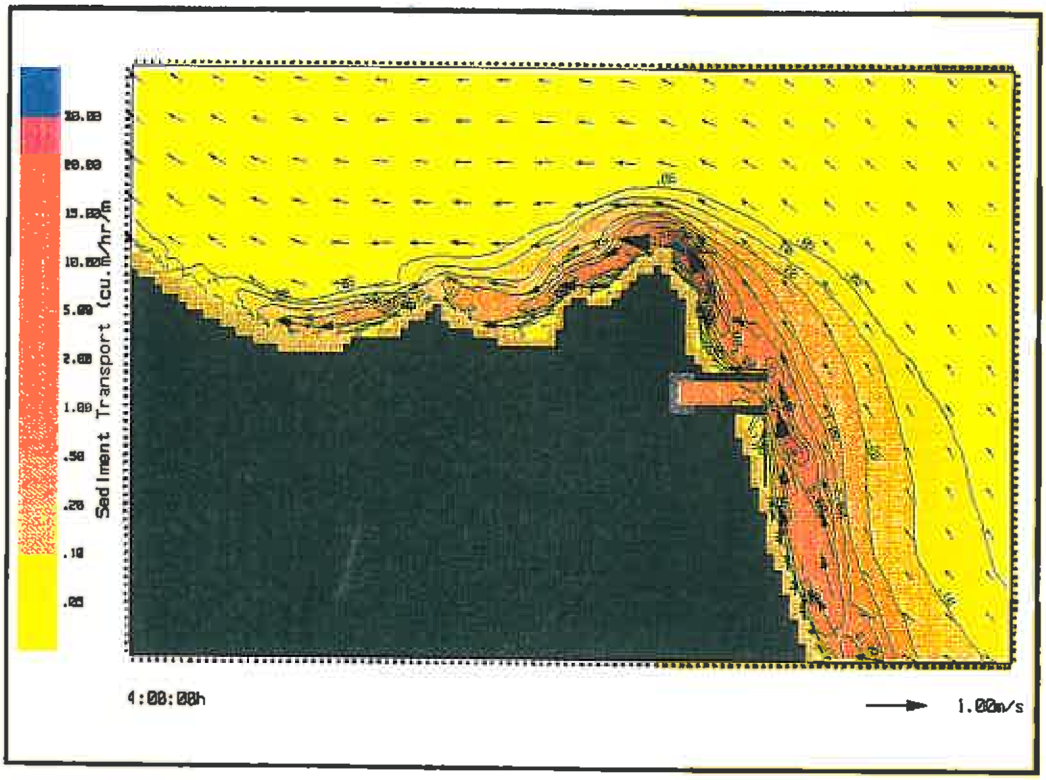


East Storm (4m9s) - Bypassing

Figure 7.3c
Impacts on Sand Transport Patterns - Easterly Storm Wave



Southeast Storm (4m9s) - Existing



Southeast Storm (4m9s) - Bypassing

Figure 7.3d
Impacts on Sand Transport Patterns - Southeast Storm Wave

For the purposes of the calculation, the wave conditions input to the channel area generate a gross transport of about 730,000 cubic metres per year, made of about 530,000m^3/year during 'modal' wave conditions up to 2.5 metres, and 200,000m^3/year during storm waves in excess of 3 metres.

Trapping efficiencies obtained from the calculations are listed in Table 7.1.

Table 7.1 Channel Trapping Efficiencies

Wave Condition	**Channel Depth (AHD)**			
	4.5	**6.5**	**8.0**	**10.0**
'Modal'	86%	97%	99%	100%
Storm	30%	60%	76%	88%

These results would indicate significant bypassing of the channel for depths up to about 7 metres, becoming substantially less and exclusively storm related for depths over 8 metres.

Approximate bypassing rates for each of the channel depths are indicated approximately as follows:

Depth	**Bypassing (%)**
4.5	30%
6.5	15%
8.0	7.5%
10.0	3%

These results are based on the assumption that bypass dredging occurs continuously to maintain these channel depths. In cases where the channel is allowed to fill with sand, greater bypassing will occur through the shallower channel.

7.5 Wave Impacts on Tweed River Entrance and Training Walls

The reduction in nearshore bathymetry by dredging of the bar and nearshore areas will mean that significantly fewer waves will break in this area and therefore the waves approaching the entrance and training walls will contain more energy. Wave propagation into the entrance will increase as the wave condition at the entrance will be increased and the channel between the breakwaters and the training walls deepened by dredging.

PWD *(1991j)* examined the impact of a permanent bypass system on the transmission of wave energy to the Tweed entrance training walls. Essentially the wave energy at the walls is a function of the magnitude of the nearshore wave height *(ie. just seawards of the bar)*, the depth over the bar at high tide, surge and wave set up and a wave breaking index *(ie. the largest wave reaching the breakwater will be the largest wave that can travel across the bar without breaking)*. It was found that bar dredging would increase the height of waves reaching the training walls, during a 5% probability storm, from 5.4 m *(existing)* to 6.8 metres. However, this was based on an assumed bar depth of -4 m ISLW *(ie. approx -5 m AHD)*. It is considered that a bar depth of -5 to -8 m AHD is possible in relation to a permanent bypass system.

For the projected situation in which the river entrance bar is largely removed, but with uncertain seabed depth and bathymetric form immediately offshore from the training walls, the maximum wave condition likely to impinge on the walls will be controlled by both:

- total water depth resulting from the dredging plus tide, storm surge and setup, and

- nearshore wave refraction which may focus or diminish the wave energy at a particular location.

In the case where dredging forms a wide, relatively flat nearshore seabed, focussing by refraction would be negligible. If shallow nearshore shoals persist, then they may focus the waves onto the training walls, although they would also limit the height of the transmitted waves.

Ignoring refraction effects allows a depth-limited design wave condition to be determined for a given nearshore depth regime. This depends on the breaker index value ($\gamma_b = H_b/d_d$).

The breaker index for a single wave is higher than that for the equivalent significant (H_s) or root mean square (Hrms) value associated with irregular wave trains. Patterson (1985) has shown that the maximum H_s value in the breaker region is about 90% of the maximum individual wave height of equivalent deep water height and period. Potential damage to rubble mound structures such as the training walls is generally related more closely to H_s than to individual waves during a design storm.

The Shore Protection Manual (CERC 1984) provides a basis for estimating maximum likely depth limited individual wave heights for a range of bed slopes and deepwater wave steepness. These have been applied for extreme storm wave conditions of deepwater steepness 0.04 to give the design breaker index values in Table 7.4.1.

Table 7.2 Preliminary Estimate of Design Breaker Index- Shore Protection Manual

Bed Slope	Shore Protection Manual Breaker Index	
	Individual Wave	Maximum H_s (approx)
0.01	0.82	0.74
0.02	0.83	0.75
0.03	0.85	0.77
0.05	0.92	0.83

Other researchers give considerably lower breaker index values (0.55-0.7), particularly for irregular storm wave conditions. Thus the SPM results would represent a conservative upper limit. Clearly, comprehensive physical modelling would be needed to assess breakwater stability for a given design situation. Nevertheless, design breaking wave conditions (H_s) of up to 7.5-8.0 metres should be considered for the case of bar dredging to -8 metres (AHD).

That wave condition is substantially greater than that assessed as the limiting stable case for the existing structures. Thus there would need to be significant upgrading of the training wall design to withstand potential future wave attack.

7.5.1 Impacts on Wave Penetration to Entrance Channel

Recorded wave data from the Tweed Regional tide gauge site indicates increased wave action at this site during the initial Stage 1(A) dredging program. This recorder site is not representative of other more exposed areas within the river channel. Hence, the absolute wave height values recorded are of limited significance. Nevertheless, these data indicate that there has been a significant and persistent increase in the height of waves propagating into the river entrance associated with the entrance dredging. The typical pre-dredging significant wave heights in the range 0.1-0.3m have clearly increased to 0.3m-0.6m. The plot of the recorded wave data is shown in Figure 7.4.

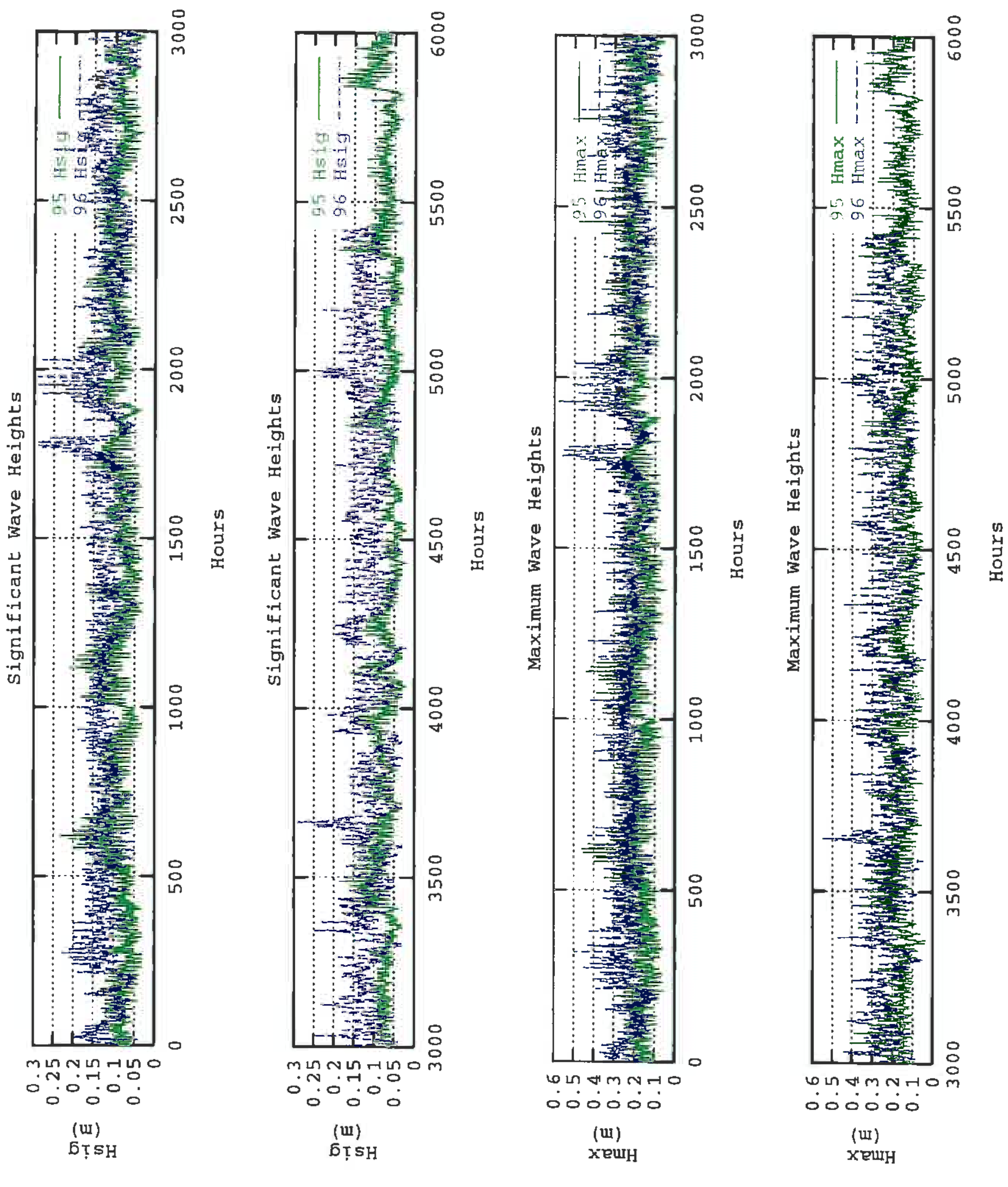


Figure 7.4
Recorded Wave Data Within Tweed River Entrance

It is likely that these increases will continue when the offshore area is dredged to its final shape. Numerical modelling of typical east-southeast swell wave penetration into the entrance channel has been undertaken for a range of existing and possible future bar and channel bathymetry situations.

These include:

- the 1995 pre-dredged bathymetry;
- the 1996 (May and December) dredged bathymetry;
- a bypassing scenario with the bar area dredged to RL-6.5mAHD, and with reduced but significant sand shoals along the southern side of the entrance channel; and
- a bypassing scenario with the bar area dredged to RL -6.5mAHD over an extensively wide area, and with minimal sand shoals near and within the river channel (depth 5.0-5.5 metres).

The results of this modelling are shown in Figure 7.5 and indicate the following:

- For the pre-dredged case, significant refraction of the waves by the bar area shoals, and also by the shoals along the southern side of the river channel, with only slight wave penetration into the river.
- For the 1996 dredged bathymetry, increased focussing of the waves by refraction over the prominent southern bar lobe. In this case, the waves focus onto the southern training wall area. For other tide levels and with the additional refractive effect of the tidal current (not modelled), the focussing may be along parts of the entrance channel itself, leading in part to increased wave heights in the river as observed.
- For the first of the bypass scenarios, increased wave penetration to the river channel, with continued refraction of the wave energy towards the southern training wall.
- For the second bypass scenario with shoals removed from the nearshore and river channel areas, a greater increase in wave penetration along the river channel. Wave height attenuation to about 20% of the condition at the head of the training walls is indicated as potentially impacting on the revetment wall at the boat basin. In the worst-case cyclone scenario with storm surge, this corresponds to waves of 1.0-1.5 metres height at that location.

8 Tweed River Tidal Hydrodynamics

8.1 Background

Previous computer modelling studies have indicated that improvement of the river entrance by deepening the entrance channel would change the tidal regime of the Tweed River estuary. Specifically, reducing the frictional resistance of the entrance relative to the quite silted equilibrium condition to which the lower estuary area has been progressively returning over recent decades would lower low tide levels and increase high tide levels in the river.

During the 1960's, the lower estuary was relatively choked with sand, with highly attenuated tidal ranges in the river (Druery and Curedale 1979). Extensive sand dredging during the 1970s changed that condition substantially, leading to increased tidal range and increased tidal prism. A by-product of that impact was greater capacity for the ebb jet at the entrance to push the river mouth bar further seaward and maintain somewhat greater depths over the bar.

The lower estuary has been slowly accumulating sand since the 1970s as the lower estuary shoals have progressively reformed with sand primarily from the beach system. This process is continuing and, if left unchecked, would probably re-establish the 1960s conditions again some 20-30 years from now.

If that were to happen, tidal ranges, tidal prism and tidal flushing in the river would once again be attenuated, and the entrance ebb jet would also be reduced. This in turn would allow the river mouth area to become even more silted until dynamic equilibrium is reached.

Hence, the existing situation is not static, but one of progressive change towards more lower estuary siltation, smaller tidal ranges, poorer tidal flushing and shallower entrance bar conditions. Approved sand extraction further upstream along the river is also predicted to have additional impacts on these processes.

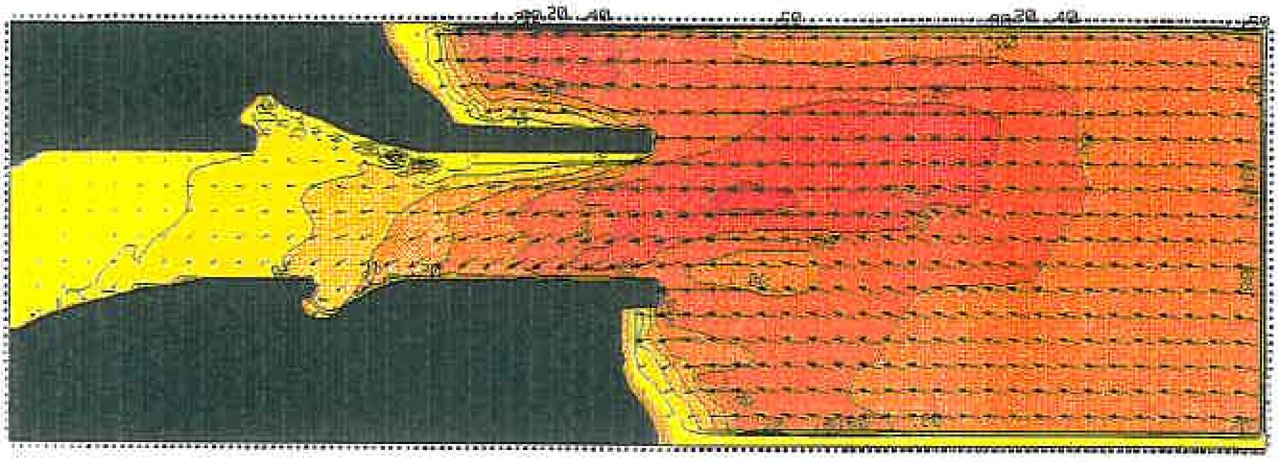
Computer modelling has been undertaken in order to quantify the present situation in the context of recent past and projected future 'existing' conditions, and to quantify the impacts of the sand bypassing project on those conditions.

Details of the modelling are presented below. Further discussions of these results is presented in the EIA document.

8.2 Computer Modelling Methodology

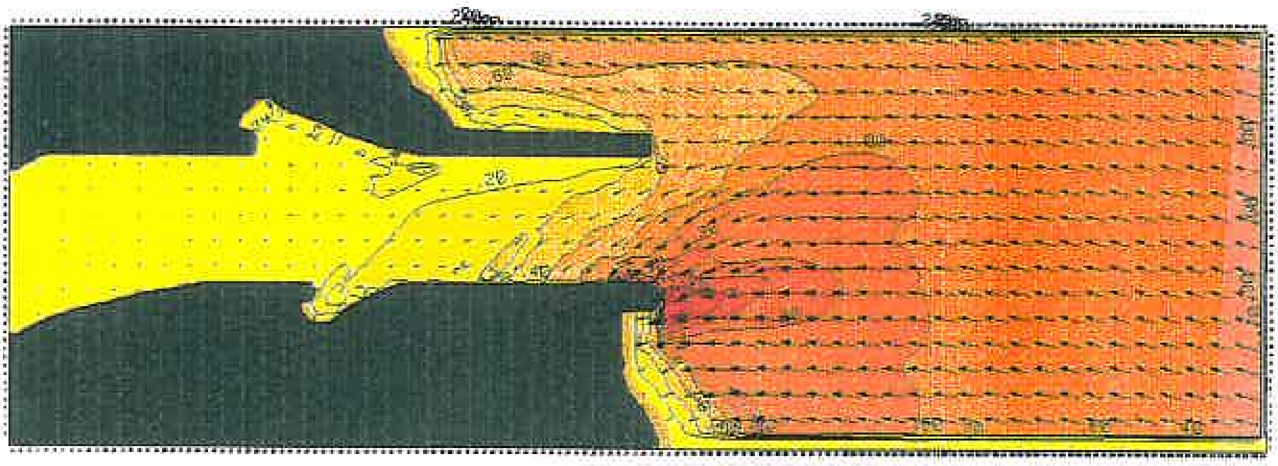
Previous computer modelling has utilised the networked one-dimensional hydrodynamic modelling software ESTRY. Bathymetric representation of the entrance area and lower estuary has been based generally on the longer term silted situation for impact assessment purposes.

One-dimensional model representation of the river mouth, although reasonable, cannot incorporate all of the processes taking place there. For the present project, more comprehensive two-dimensional representation of the river entrance and nearshore coastal area has been adopted. In that way, specific bathymetric configurations and dredging status can be represented, and effects on both the river tidal regime and nearshore current patterns reliably determined.



Pre-Dredging Entrance

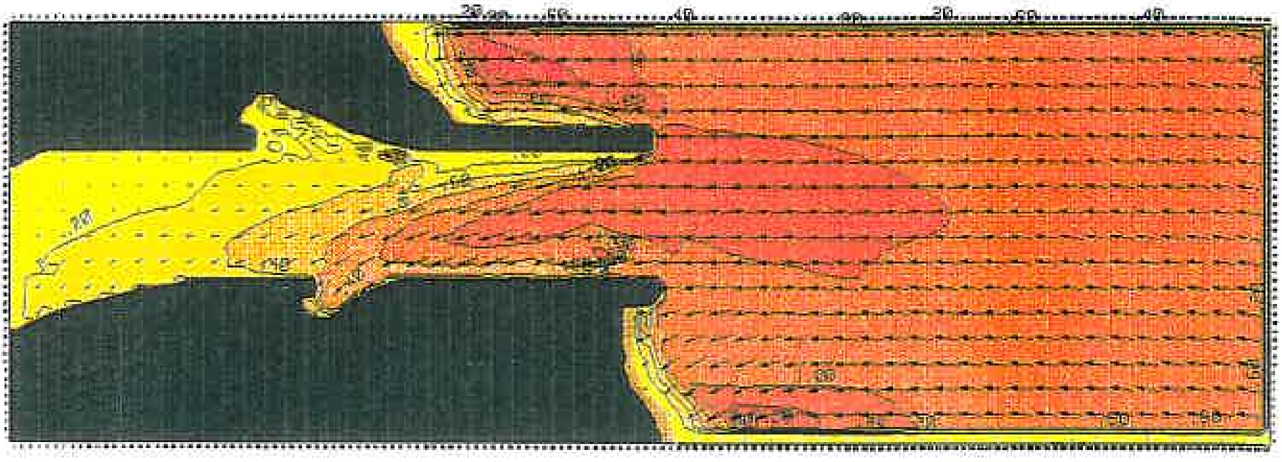
→ 1.00m



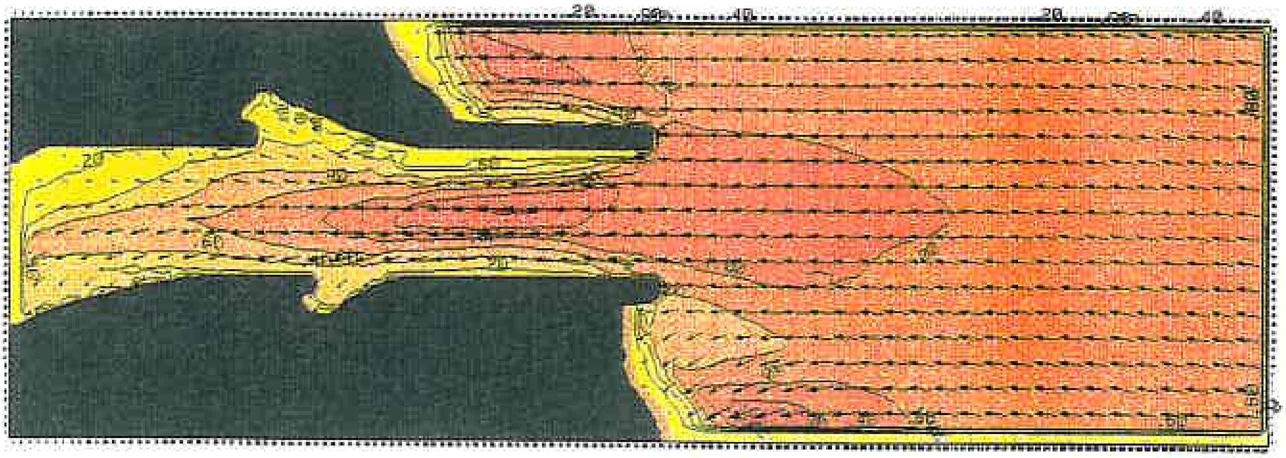
Dredged Channel (May 1996)

→ 1.00m

Figure 7.5a
Wave Penetration to Tweed River Entrance - Existing Scenarios



Dredged Entrance Bar Area



Dredged Bar and Entrance Channel

Figure 7.5b
Wave Penetration to Tweed River Entrance - Dredged Entrance Bar Scenarios

The TUFLOW/ESTRY modelling software has thus been used. A dynamically linked 2D/1D model interface was established near the Tweed Regional Gauge, some 500 metres upstream of the training wall heads. The Tweed River tidal model, previously calibrated to 1992 data for the Regional Gauge and at various locations along the river was thus utilised.

Surveyed seabed bathymetry both between the training walls and in the nearshore river mouth area was input to the 2D model section as appropriate for each scenario tested. Corresponding changes to the river bathymetry associated with dredging and/or lower estuary shoal changes are also able to be made.

8.3 Scenarios Modelled

River and bar bathymetry conditions tested in the linked model have been established to represent the present day, recent past and projected future scenarios for the pre-dredging, dredged entrance channel and future bypassing situations. In that way, the effects of the bypassing may be more readily understood in the context of the ongoing evolving existing situation.

Specific scenarios tested in the modelling are as follows:

Pre-Existing Case Scenarios

Case 1:	1992 - No river or bar dredged
Case 2:	Case 1 plus Areas, A, B & C dredged
Case 2A:	Case 2 plus Area 5 dredged - existing estuary shoals
Case 2B:	Case 2A plus lower estuary shoaled by 250,000m^3
Case 2C:	Case 2A with 200,000m^3 removal from estuary shoal

Improved River Entrance Scenarios

Case 3:	Case 2 plus entrance channel dredged (per May 1996)
Case 4:	Case 2 plus entrance bypassing
Case 5A:	Case 2A plus entrance bypassing
Case 5B:	Case 2B plus entrance bypassing
Case 5C:	Case 2C plus entrance bypassing

The locations of Areas A, B and C and Area 5 are shown in Figure 8.1. A description of the dredging proposed for those areas is presented in the respective EIA documents for each activity.

The dredged entrance channel scenario (Case 3) is based on the survey of 24 May, 1996. The shoaled lower estuary scenario provides for the additional input of about 250,000 cubic metres of sand from between the Regional Gauge and Letitia Reach.

For all entrance bypassing scenarios, the nearshore entrance channel and bar area has been set at RL -8.0mAHD over a wide area, and thus represents the upper limit of likely hydrodynamic impacts on the tidal regime of the estuary. In reality, deeper or shallower channel and bar depths may occur from time to time depending on the bypass system adopted and its mode of operation.

8.4 Model Results

The results of the modelling are presented in time series form in terms of water levels and flow discharge rates at various locations throughout the river system for each scenario tested. These results are presented in Appendix E. Discussion of the results is presented in Sections 4.4 and 7.4 of the EIS report.

9 Sand Bypassing Considerations

9.1 General Considerations

The proposed bypassing project provides for primary sand discharge at Point Danger, with some placement permitted also at Kirra (15%) and Duranbah (10%) as required. This will place the sand past the influence of the Tweed River, aimed at preventing the continued development of the entrance bar and ensuring that the sand is supplied effectively into the Gold Coast beach system.

The longshore sand transport modelling undertaken confirms the previously assessed average net transport rate of about 500,000 cubic metres for this beach system, and provides quantified information on its spatial and temporal variability. The basis of the Deed of Agreement with regard to the average bypassing rate (500,000m^3/year) and variability from year to year is thus confirmed independently.

9.2 Impacts on Beach System

9.2.1 Gold Coast Beach Amenity

The modelling confirms that the sand bypassing should achieve the objective of delivering sand to the beaches at an average rate which matches the net longshore transport capacity. This will provide long term dynamic stability to the beach system.

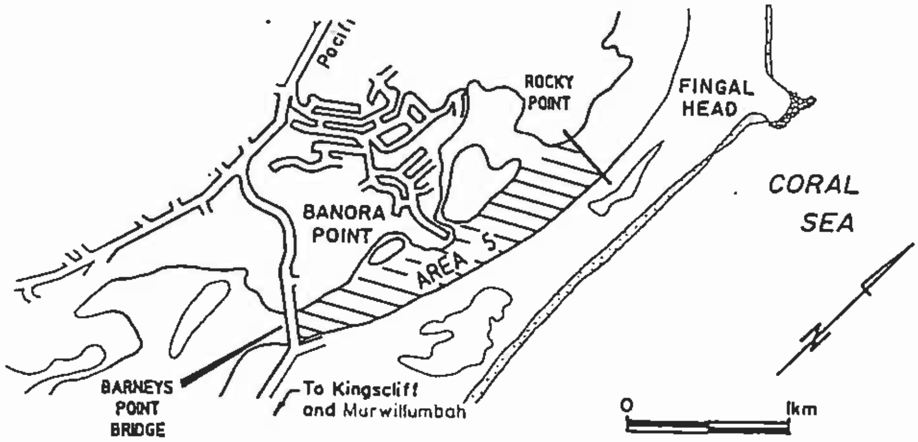
The bypassing will not, and is not intended to, prevent short to medium term fluctuations in the width and amenity of the beaches. The extent of such fluctuations could be (order of) 50 metres in those areas immediately downdrift of the headlands. Similarly, the nearshore sand shoals and bars will change constantly in response to changing wave conditions in the natural manner.

Despite that, the flexibility in sand discharge locations, quantities and timings provides some capability to minimise adverse beach loss from time to time. This may take the form of direct placement at Kirra, Snapper Rocks or Duranbah when needed to offset erosion, or discharge of sand at places and times most beneficial for maintenance of the longshore supply.

In particular, the modelling has shown that prolonged periods of northeast waves, usually occurring during September to December can deplete the sand reserves at Point Danger. This occurs because, in such conditions, sand is transported both southward towards Duranbah and westward past Rainbow Bay, causing a net loss at Snapper Rocks. It will be feasible to discharge directly to Snapper Rocks at such times to maintain the sand supply.

The beaches at Rainbow Bay, Greenmount and Kirra will continue to exhibit most variability because they are subject to greatest variations in longshore sand transport capacity relative to the supply from the beaches immediately updrift. The bypass system discharge strategy will maximise the sand supply and minimise this variability.

It must be recognised that sand transport along the beaches is a combination of longshore and cross-shore migration of sand. Storms will continue to take sand from the beaches and place it nearshore where it forms part of the longshore supply and helps to maintain the nearshore bars and shoals which form the best surfing conditions. Regular transfers of sand will occur to and from depths up to about 4 or 5 metres, with less frequent transfers out to 6 to 8 metres.



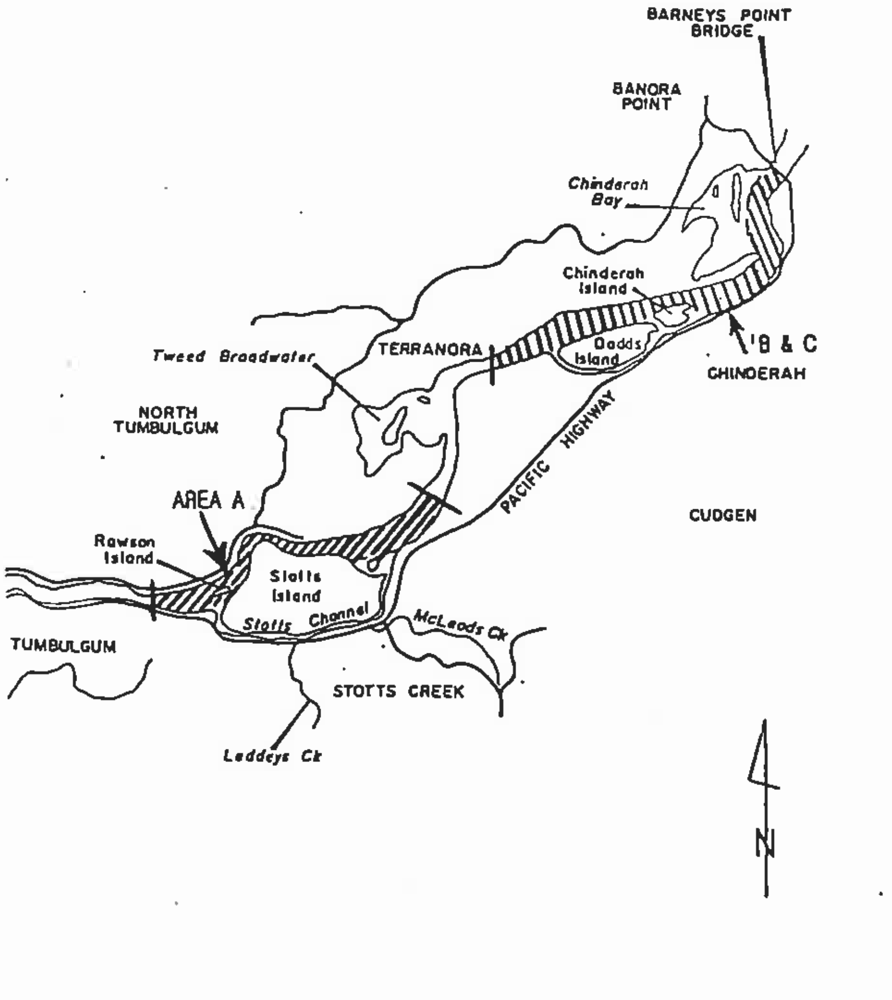


Figure 8.1
Location of Dredging Areas A, B & C and Areas

Cross-shore transport of sand also moves sand shoreward from deeper water onto the nearshore bars and eventually onto the beach during normal swell conditions. Thus, the beaches will fluctuate and recover naturally over time without interference provided the required supply of sand is maintained.

9.2.2 Sand Discharge at Snapper Rocks

Snapper Rocks (or a location in the vicinity) is designated in the Deed of Agreement as the predominant discharge site. There is some flexibility in the precise location of the discharge at any time, both along a zone in the general vicinity of Frogs Beach, Point Danger, on the eastern (ocean) shoreline and at Snapper or Marleys Rocks immediately west of the Point.

The sand transport modelling undertaken for this study supports this discharge strategy as being the most suitable for introducing the sand into the longshore transport system of the Gold Coast beaches. A minimum of 75% of the discharged sand would be placed directly in this area.

The response of the nearshore bathymetry and sand transport patterns in this region will be dependent on both the discharge strategy and the action of the waves in redistributing the placed sand across the active profile.

The computer modelling of longshore transport processes has shown that essentially all of the longshore sand supply to the Queensland beaches occurs at Point Danger in a relatively narrow band within the -8 metre (AHD) depth contour. Upcoast transport occurs seaward of that depth during the larger storm events, but this tends to be balanced by the persistent downcoast transport influence of the East Australia Current. The zone of predominant longshore transport is somewhat wider in the area north of Snapper Rocks, extending out to about 10 metre contour.

Thus, it is concluded that all of the bypassed sand placed such that it is confined within the 8 metre contour in the Frog's Beach area would be transported by the natural wave/current processes to the Gold Coast beach system. This is illustrated in Figures 9.1 and 9.2.

Placement of the sand in that area to achieve a suitable distribution into the sand transport system of the Gold Coast beaches could thus be achieved by one or more of the following procedures:

- discharge directly from the shore, allowing the waves and currents to redistribute the sand both alongshore and seaward;
- discharge from a trestle structure or detailed platform within (about) the 6 metre contour; or
- bottom dumping from mobile hopper dredgers within (about) the 6 metre contour.

The placement of sand at Point Danger will lead to a number of beneficial impacts including:

- potential development of a sandy beach there, at least from time to time;
- inducement of a more persistent sand supply north from the discharge area past Point Danger to Snapper Rocks, Rainbow Bay and beaches further downdrift;
- the opportunity to achieve cost-effective placement of sand into the beach system in a manner which closely mirrors that of the natural sand transport system (with operational variability to enhance beach condition), thereby ensuring redistribution to the southern Gold Coast beaches and nearshore shoals appropriately; and
- there will be natural wave-induced redistribution of sand placed in the upper profile area to somewhat deeper water, both at the discharge site and as the sand moves away to the north.

There could be sand available in this area to move southward to Duranbah during northeast sector waves, predominantly during the period September to December, thus helping to preserve the size and amenity of that beach.

As an additional or alternate strategy, sand placement could be redirected to Snapper (or Marley) Rocks. This would assure continued persistent supply to the Gold Coast beaches at times when the net transport capacity at Point Danger is such that little or no supply would otherwise occur.

A predominance of sand discharge directly to the beaches west of Snapper Rocks would also ensure effective delivery of all of the bypassed sand to the Gold Coast beaches. However, in such cases, care would need to be taken to distribute the sand over a somewhat wider area north of Snapper Rocks across the whole active transport zone. This would be needed to prevent excessive supply at Snapper Rocks and to prevent longer term denudation of the deeper water bathymetry which controls wave refraction to the adjacent beaches.

Placement of sand in this area could be achieved by direct discharge from the shoreline or as bottom dumping from hopper dredgers. The majority (about 75%) of the placement should occur within the 6 metre contour, with the remainder acceptably placed out to (about) 8 metres, to ensure that all sand is contained within about the 10 metre contour.

However, placement in this area could present adverse impacts with respect to:

- direct physical interference with recreational surfing activities and beach amenity; and
- excessive turbidity plumes in the surf areas widely used for surfing.

Thus discharge predominantly to the Frog's Beach area, Point Danger, is recommended as the preferred option, with discharges to Snapper/Marley Rocks taking place only as part of special-purpose exercises when specifically needed and planned to minimise such adverse impacts.

As well, the profile bathymetry in deeper water offshore from Point Danger and the Gold Coast beaches may experience progressive depletion due to the changed nature and path of sand supply. If this occurs, the change will be slow, being related to the small differentials in longshore transport in depths where substantial quantities of sand will be available to maintain a natural supply.

Any such depletion over the longer term may cause some adverse effects in the form of changes to southeast wave refraction patterns and reduced sand supply in deeper waters to the north.

Survey monitoring of the discharge region and areas offshore and to the north should be undertaken as part of the sand discharge management strategy. Remedial placement activity could be undertaken from time to time if required. Any such reactive action should be assessed on a long term average trend basis, and not as response to short term transient changes.

9.2.3 Duranbah

Over time following implementation of effective sand bypassing, the sand supply into Duranbah will diminish. The project provides for placement of up to 10% (50,000m^3/year) of the bypassed sand at Duranbah.

Duranbah beach will thus tend to retreat and the nearshore profile become deeper to establish a new equilibrium with the altered longshore transport regime. Sand presently in the river mouth bar area will

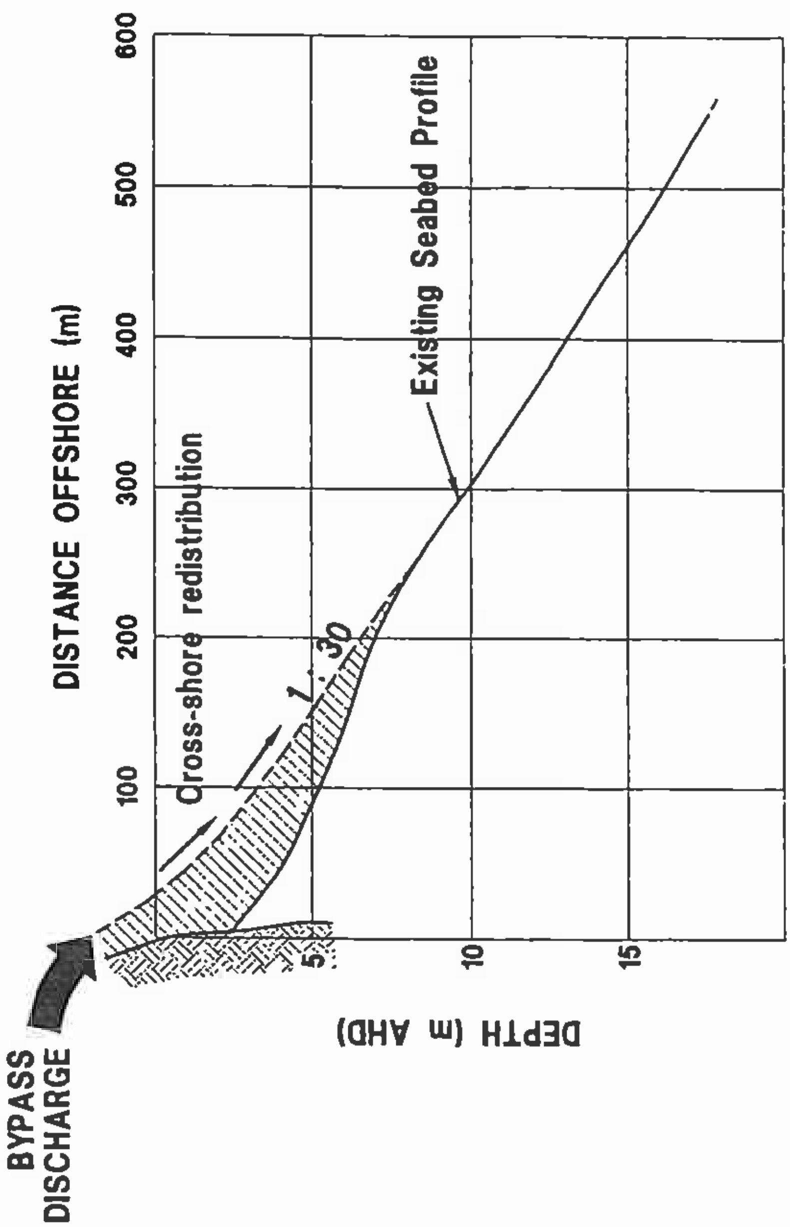


Figure 9.1
Cross-shore Re-Distribution of Sand Placed at Frogs Beach

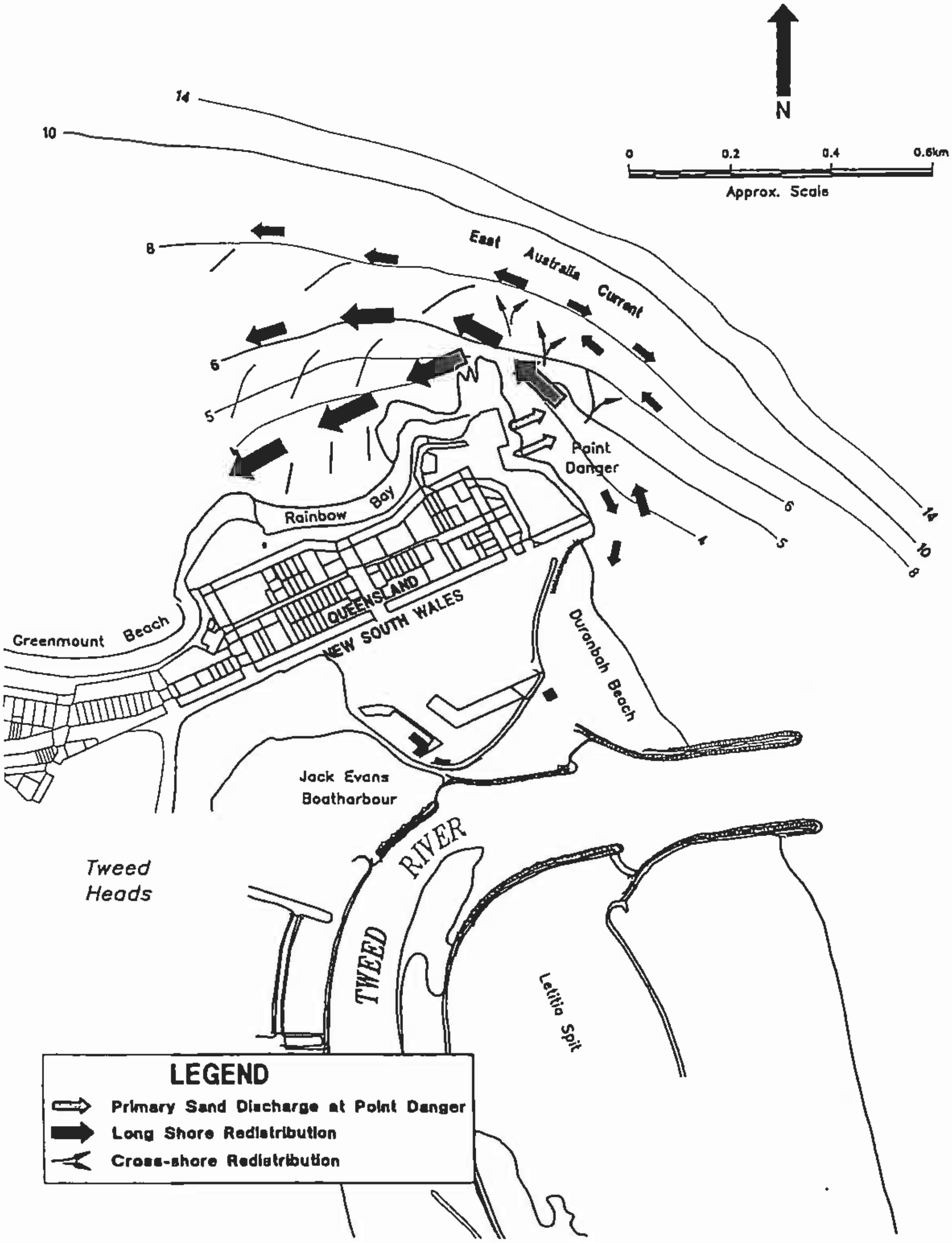


Figure 9.2
Spatial Re-Distribution of Sand Placed at Frogs Beach

move both onshore and northwards in this process, and will not be replaced by supply from the south unless there is 'leakage' of sand across the channel.

The longshore transport modelling indicates a significant short term reduction in transport with removal by dredging and wave/current action of the shallow bar area offshore from the beach. The zone of storm wave transport will move in closer to the beach. The overall longshore transport capacity of the beach will diminish progressively over time as the beach retreats.

The beach shape will tend towards that of a pocket beach held between the northern training wall, and the headland. The eventual beach shape will depend somewhat on the sand discharge strategy. One-off placements of larger quantities of sand each year may create nearshore shoals which take some weeks or months to disperse. Continuous discharge at a low rate will have only minor effects on the beach shape.

The beach and dune system will re-establish a new equilibrium condition over time. The amenity of the beach in its new location and alignment should be similar to that which exists at present. It should be noted that this was a period of relatively high wave climate and that the retreat rate decreased with time.

Surfing conditions will change in response to the changes in the beach shape and nearshore bathymetry. The separate report by Dr. J Walker on surfing impacts outlines details of these changes, and considerations for mitigation.

9.2.4 Impact of Heavy Mineral Extraction on Beach Processes

The Gold Coast beach sand is made up of three classes of mineral; light, intermediate and heavy. The light minerals have specific gravity of about 2.65. The heavy minerals have specific gravity ranging from 4.5 to 5.0, and are significantly finer than the light minerals. On the beaches of the Tweed Coast and Gold Coast, white sands dominate, these being made up of predominantly light minerals *(silicates)*. Zircon, rutile and ilmenite make up 97% of the total the heavy minerals. The lighter white silica sands are coarser and rougher in nature than the heavier black minerals. When mined, the heavy minerals are separated from the light minerals and processed into pure forms for sale, and the lighter sand components are returned to the beach system. In the past, mineral sand mining has occurred extensively on the Southern Queensland and Northern NSW coasts *(Logan, Tomlinson and Robinson, 1995)*.

Patterson Britton & Partners *(1989)* completed a comprehensive assessment of the commercial potential of mineral sands in the Tweed littoral system. The study found that the economic mineral sand component of the active marine sands of the lower Tweed and entrance bar varies from 0.07%wt to 0.52%wt of the total body, with an average of 0.2%wt.

The fundamental concept behind most relationships (for example Bagnold, 1963) to quantify sediment transport by wave/current action involves the following principle:

(i) The wave orbital motion stirs up the bottom sediment and puts it into oscillatory motion.

(ii) The bottom sediment, once stirred up by the waves, can be moved in the direction of any superimposed current no matter how small the current is. These currents can be either currents generated by the wave motion itself or currents generated by other agencies such as tides, winds, density differences, etc.

The rate of transport of sand by waves and currents in the surf zone depends on both the grain size and density of the sand material, involving also the fall velocity. For fine to medium sand , theoretical equality of transport potential is achieved when the sand grain size is inversely proportional to the immersed relative density $(S-1)^{1.6}$, where S is the specific gravity of the sediment. Thus, heavy minerals will be transported with similar potential to that of the rest of the beach sands provided they are significantly (about 0.25-0.4 times) finer than the silica components. Natural sediment sorting has established that situation.

In the swash zone of the beach, different factors become dominant. In particular, sedimentation processes are determined by the uprush and backwash of the waves and the associated transport, deposition and resuspension of the sediments. This lends itself to a heavy mineral transport theory called the "Swash Zone Theory" *(Logan, Tomlinson and Robinson, 1995)*. The theory states that during severe wave conditions *(storms)*, waves transport sediments as they run up the beach. At the top of the wave path, the sediments are deposited. As the backwash moves down the beach, it accelerates and picks up the lighter materials *(silicate sands)* deposited on the washup, leaving the heavy minerals which are less readily remobilised. The lighter materials are carried offshore to form bars and the heavy mineral sands concentrate onshore.

Heavy minerals can concentrate up to 60% on the beach surface as a result of this swash action. This formation of seams of concentrated heavy minerals is reversible *(Smith, 1988)*. As the storm decays the swash length deceases and begins to undermine the mineral seam, remixing with the light sand and carrying it back to the surf zone as grey streaks across the beach. Under normal to moderate surf this mineral concentrating and remixing occurs continuously. Only under cyclone events do the seams become permanent as they form much higher up the beach. These seams become buried by windblown sand during calm weather and are able to maintain their coherence *(Smith, 1988)*.

These lenses of mineral have been considered to be important in beach stability. During extreme surf conditions the upper layers of light sand were thought to be eroded until one of the heavy mineral seams is reached and then erosion ceases due to the consolidated stable nature of the seam. However, at present there is no substantiated evidence to suggest that this actually occurs. Rather, the process of ongoing reworking and sorting of the component sands probably continues, with the lighter silicate material being taken offshore to satisfy the bar formation demand of the wave/current action. Any tendency to reduce that process would tend to increase the incident wave energy at the beach and thus increase the erosive potential.

The rate of windblown sand transport in the beach and dune system is related to a number of factors including the wind velocity, sand size and density, and the wetness of the sand surface and its geometry. Substituting the relative changes in sand density, porosity and size into Bagnold's (1941) equation for wind transport of sand, indicates that demineralisation will result in a 0.25% increase in windblown sediment transport. This is considered to be of no significance.

Thus, the removal of heavy minerals would have no impact on the sand transport regime of the beaches in the study area. However, the volume of sand involved in such removal would be finite (ie. of the order of 1,000-1500m^3 per year) and it would be necessary to replace the removed sand with an equivalent volume of silica sand, of appropriate grain size to maintain the long term average net supply of sand to the southern Gold Coast Beaches.

9.3 Impacts on River Entrance Navigation

A principal objective of the bypassing is to allow maintenance of deeper water navigable conditions over the river entrance bar. A by product of the deeper channel will be somewhat reduced tidal currents over the bar, this was confirmed in the two-dimensional modelling. These improvements should benefit navigation in the bar area seaward of the training walls.

However, there will also be changes in both exposure to wave action and tidal currents within the limits of the training walls affecting river entrance navigability. As such, navigation conditions will be changed by the bypassing, and the manner in which the entrance is negotiated at both ebb and flood tides will need to be reviewed to suit the changed conditions.

Wave action changes could be quite complex and will depend significantly on the nature of dredging of the entrance channel. Specifically, wave propagation to the training wall area may be either focussed or dispersed by refraction, depending on the shape of the nearshore bathymetry.

The modelling shows that there will be a slight increase in tidal current speeds in the area between the training walls. The speed of these currents will depend on the amount of sand accumulating there from time to time, although regime equilibrium conditions are likely to prevail in the longer term.

The interaction of waves and currents within the entrance channel is an important factor affecting navigability. In particular, strong ebb currents may increase wave heights and steepness to an extent that navigability is adversely affected. Some comments have been made following the Stage 1(A) entrance dredging that the wave/current interaction has worsened, at least for some craft.

Both the water level monitoring in the river and the computer modelling described herein have shown that the change in tidal flow caused by the dredging is minimal. The bypassing would have quite minor additional impact.

It is feasible that wave conditions within the entrance have been changed by the dredging such that navigation has been affected. Wave monitoring at the Regional Gauge confirms that typical (modal) wave heights in the river increased following the dredging This could be primarily related to:

- reduced attenuation by wave breaking over the bar,
- reduced attenuation by changed refraction over the bar, or
- increased wave focussing by changed refraction over the bar.

The shape of the southern lobe of he dredged entrance bar is such that focussing of waves from some directions might be expected. Such conditions are unlikely to persist following implementation of the bypassing, but may need monitoring with a view to optimising the bypass operations to minimise any adverse effects.

9.4 Beach System Management Strategy

9.4.1 General Considerations

management in order to maximise the effectiveness of the bypass system and the benefits to the beach system. Broadly, bypass system strategies for effective operation and beach management relate to:

- sand dredging at the bypass intake area, and
- sand discharge to the beach system.

Ideally, bypassing operations of dredging and discharge seek to parallel the natural longshore transport patterns as much as practicable. These activities are inextricably linked, and yet conditions at the dredging area may not always match those at the discharge point, and discontinuities in sand supply and demand will occur from time to time.

Thus operational and monitoring strategies need to be put in place to ensure effective bypassing over the longer term.

9.4.2 Sand Dredging and Point Danger Discharge Strategy

The strategy adopted at the intake area will depend intimately on the type of system adopted. Specifically, a fixed trestle system depends on sand being brought to the intakes by the prevailing waves and currents. Thus, in times of low waves, little sand will feed to the system and only small quantities may be bypassed. Alternatively, a mobile intake system (hopper dredge or jack-up platform) may be moved over a wide area and collect sand regardless of the prevailing transport conditions.

In the case of the fixed intake system, low waves and supply at the intake will coincide with low waves and demand at the discharge. However, there would be a corresponding requirement to match the transports during high energy waves if no reserve of sand is built up at the discharge point during calmer weather. This has proven to be difficult at other bypass plants, where system shutdown usually occurs in storm conditions.

Thus, it is desirable that the bypassing seeks to dredge and bypass more sand in calmer 'modal' conditions to build a discharge reserve at Point Danger to cater for the storm periods. This can be achieved readily for mobile dredging plant.

A key issue in this with regard to the fixed trestle option is that the sand transported to the intakes is the gross (upcoast and downcoast) transport, which exceeds the net rate of 500,000m^3. This is more so at Letitia Spit than at any other location along this beach system.

The modelling indicates an existing typical gross transport of 750,000-1,000,000m^3/year at Letitia Spit. This can be expected to decrease somewhat as the system changes the sand transport regime by depletion of the bathymetry in the entrance channel area. Nevertheless, even for fixed trestle system, if properly designed and operated, the opportunity would be there to dredge the full quantity to build and maintain the discharge reserve at Point Danger.

It is important to note that the experience at the Gold Coast Seaway cannot be applied directly to the Tweed in this regard. At the Seaway, the southern training wall acts as a partial sand trap for sand leaking past the system, providing a mechanism for return of the sand in northeast waves. This will not be the case at the Tweed. Hence, a fixed trestle system at Tweed will need to be designed and operated to collect upcoast transport more efficiently, when it occurs, to minimise leakage into the entrance channel.

9.4.3 Sand Discharge in Northeast Waves

The downcoast (northeast wave) component of the gross transport at Letitia Spit coincides with downcoast transport at Point Danger, but upcoast transport along Snapper Rocks and Kirra. Thus a choice will need to be made about the discharge during such conditions.

Periods of local northeast sea waves tend to occur predominantly in spring (September through December). Usually, there is an underlying southeast swell occurring at the same time. This period in particular will need monitoring of the Point Danger sand reserve and adjacent beach conditions to determine the most appropriate discharge location(s).

At this stage, it is sufficient to ensure that provision be included in the bypass system design for direct discharge to both Snapper Rocks (or Marley Rocks) and Kirra as alternatives to the primary discharge at Point Danger (Frogs Beach).

9.4.4 Storm Period Operation

It is recognised that operations during storms will be difficult. This is because of adverse wave and weather together with the prevalence of debris in the sand which can cause problems with the bypass pumping. Nevertheless, the objective of the system should be to mimic the natural sand transport regime as far as practicable. The extent to which operation is necessary during storms will depend on a number of factors, predominantly -

- the ability to maintain a reserve of sand at Point Danger during normal wave conditions to act as a supply for the beach during storm periods;
- the degree of acceptance of beach variability associated with short to medium term deficits of supply at Point Danger;
- the size and efficiency of the dredged sand buffer at the entrance channel area, and its capacity to accommodate siltation without causing navigation problems; and
- the extent of overall 'leakage' of sand through the system, which will tend to occur during storm periods.

The design and operation of the system must recognise that beach and nearshore sand is moved offshore to deeper water during major storm events. The zone of predominant transport correspondingly moves further offshore. One or two nearshore bars may form. The historical survey evidence suggests that these bars may extend out to existing water depths of 7-10 metres, some 500 metres offshore from the existing beach.

For a fixed trestle system, consideration will need to be given to the most cost-effective means of collecting this longshore transport occurring such distances offshore. Extension of the trestle to such lengths would be costly and of little benefit in normal waves. Accompanying dredging by mobile plant (hopper dredge) immediately following storm events may be feasible.

9.4.5 Day/Night Operations

The design of the Seaway bypass system, particularly with regard to its remote location and the updrift beach alignment relative to the extent of the training walls, is such that the option is available to concentrate the bypass dredging during night hours to minimise power costs. This may not be the case at the Tweed for the following reasons:

(i) The Tweed training walls will not act as a downdrift sand trap to minimise leakage of sand through the system. Thus sand dredging may need to be more continuous;

(ii) The Letitia Spit alignment is such that upcoast transport occurs for east waves, more so than at the Seaway; and

(iii) The Tweed site is relatively close to residential areas and noise nuisance will need to be avoided.

Noise management issues are discussed in Sections 7.6 and 8.6 of the EIS. It is understood that specific design criteria will need to be specified for the on-land pump station to attenuate the noise if night time operations are to be undertaken.

To ensure effective operational efficiency, it is recommended that provision be made in the initial system design and costing for flexibility in the operations to include both night and day time bypassing as required.

9.4.6 Kirra Discharge

The project provides for an average of up to 75,000m^3/year (15%) to be placed directly at Kirra. This is intended primarily to restore short term erosion of the beach there to maintain its recreational amenity. As such, direct placement to the subaerial beach may be needed.

Alternatively, the sand could be placed into the nearshore longshore transport system. This could provide benefit to the surf break along Kirra Point, while also providing a less direct and immediate sand supply to the beach itself. It should be noted however, that such nearshore placement may not result in significant benefit to the beach if it remains within the nearshore bar system.

Clearly, monitoring and flexibility are needed to optimise this activity in the longer term.

9.4.7 Duranbah Discharge

As for the Kirra discharge, provision exists for direct discharge of sand to Duranbah. Up to 50,000m^3/year (10%) may be placed there as the long term average.

It has been noted that Duranbah will be altered by the bypassing project, with shoreline retreat and depletion of the nearshore shoals. The beach will tend to develop 'pocket' beach characteristics, but should retain good recreational amenity. Surfing conditions are predicted to change significantly.

Sand discharge to Duranbah may be utilised to either:

- influence the alignment and shape of the beach itself, or
- assist in providing good surfing conditions.

In both instances, the beach would benefit from the placement of sand. However, the discharge strategy will be different if the focus of the discharge relates to surfing and not beach maintenance.

Walker (1996) has identified two discharge options for surf amenity enhancement. These are:

- discharge off the northern training wall to provide an alongshore bar aligned suitably for good surf, or
- discharge to a location in about 5 metres water depth about mid-beach to create a shoal suitable for surf wave formation.

Discharge off the training wall is the less expensive of these options. Creation of an isolated mid-beach shoal would require special design and operational action and cost.

Walker estimates these options as of limited duration (up to several months), although it is not feasible to predict the behaviour of such shoals or bars with any reliability. Hence, planning of a specific strategy based trial and error may be needed.

A first step in the Duranbah sand discharge strategy is to identify the primary objective and to decide on priorities for action based on likely costs and benefits.

9.4.8 Reactive Monitoring

Based on the above considerations, it is essential that a reactive monitoring program be implemented as part of the bypassing project. Such monitoring may include:

- sand bypass quantity measurement
- simple beach and surf condition observations
- detailed beach and bathymetric surveys
- directional wave recording
- continued assessment of the longshore sand transport regime
- community feedback with respect to beach/surf conditions, entrance navigation, and environmental impact.

The key objectives of the monitoring should include:

(i) verification of sand quantities bypassed

(ii) verification of entrance channel depths, and defining of its location and alignment from time to time

(iii) identification of sand discharge requirements in terms of locations and quantities to maintain beach/surf amenity.

(iv) Correlation of sand dredging and discharge with longshore transport rates to progressively optimise the operation procedures.

It should be understood that it is neither feasible nor cost-effective to undertake reactive works in response to short term variations in the beach system. Substantial variability of beach widths and surfing conditions is an inherent natural feature of these beaches.

While it may be practical to undertake placement of sand at Snapper Rocks, Kirra or Duranbah from time to time, such action should be planned in the context of the longer term operation of the system. Similarly, any special maintenance activity such as dredging of the bar and entrance channel area or placement of sand in deeper water off Point Danger need not occur as short term emergency works, if the bypassing system will act to overcome such problems in the longer term.

Hence, a long term monitoring program aimed at identifying the medium to longer term trends of change needs to be designed and implemented.

10 References

Bagnold, R.A. (1941). The Physics of Windblown Sand and Desert Dunes. Methuen, London.

Bagnold, R.A. (1963). Mechanics of Marine Sedimentation. In *Hill M.N. (ed). The Sea*, New York, Interscience, Vol 3, pp 506-582.

Bakker, W.T. (1968). The Dynamics of a Coast with a Groyne System Proc. 11th Conference on Coastal Engineering London.

Battjes, J.A. (1974). Computation of Set-Up, Longshore Currents, Run-Up and Overtopping due to Wind-Generated Waves: Delft University of Technology Communications on Hydraulics, number 74-2. Department of Civil Engineering.

Bijker, E.W. (1968). Littoral Drift as A Function of Waves and Current: Delft Hydraulics Laboratory publication number 58, 1969. An extended summary also appears in: Proceedings of the 11th Coastal Engineering Conference: London, 1968.

Coastal Engineering Research Centre. (1984). Shore Protection Manual, 4th Edition. U.S Army Corps of Engineers, Fort Belvoir, Virginia.

Cresswell, G. (1987). The East Australian Current. CSIRO Marine Laboratories Information Sheet No. 3, June 1987.

Logan, K.L., Tomlinson, R.B. and Robinson, D.A. (1995). Heavy Mineral Concentration on Beaches. Proc. 12th Australian Conference on Coastal and Ocean Engineering.

Longuet-Higgins, M.S. (1972). Recent Progress in the Study of Longshore Currents. In *Waves on Beaches and Resulting Sediment Transport*. Edited Meyer R.E., New York Academic Press.

Patterson, D.C. (1976). Beach Profiles. Unpublished research report for post-graduate Dip. H.E (Delft), IHE, Delft.

Patterson, D.C. (1986). Low Cost Visual Determination of Surfzone Parameters. M.Eng.Sc. thesis, Dept Civil Engineering, University of Queensland.

Queensland Department of Environment and Heritage (1994). Wave Data Recording Program, Brisbane Region, 1976-1994. Conservation Data Report No. W09.2, August 1994.

Roelvink, J.A. and Murray, R.J. (1992). Gold Coast, Queensland, Australia - Southern Gold Coast Littoral Sand Supply. Report H85, Delft Hydraulics, Delft, Netherlands.

Ross, C.H. (1988). Operational Numerical Weather Prediction System, Documentation Paper 5.1, The Operational Wave Model. British Meteorological Office, December 1988.

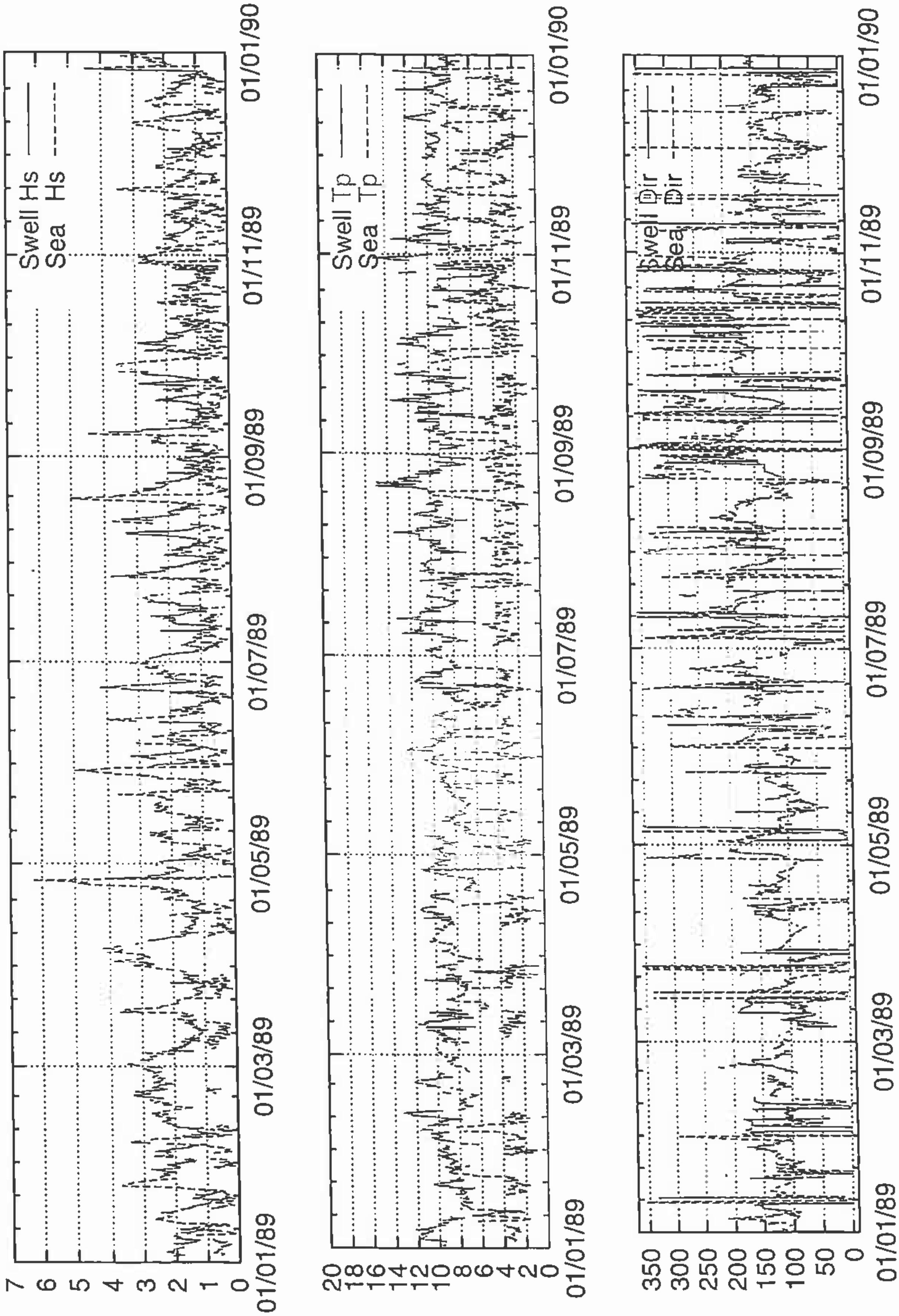
Smith, A.W. (1988). The Behaviour of Heavy Metals in a Natural Beach. Gold Coast City Council report.

Smith, A.W. (1994). Gold Coast Research and Development Report No. 200: Beaches and Tourism - An Example of the Results of a Dramatic Beach Erosion Episode. (unpubl.).

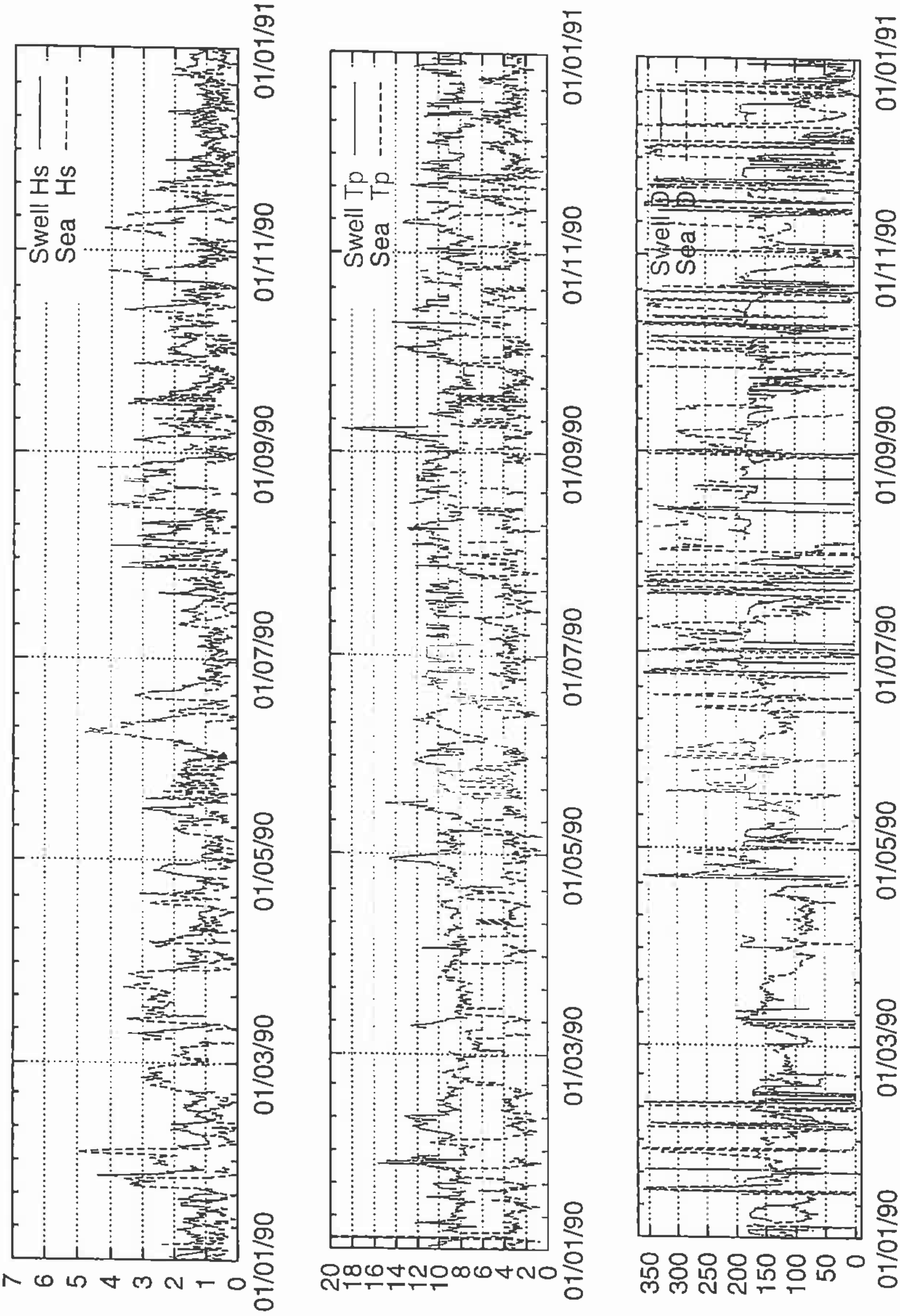
Swart, D.H. (1974). Offshore Sediment Transport and Equilibrium Beach Profiles. Delft Hydraulics Laboratory Publication No. 131.

Van Rijn, L.C.(1990). Handbook - Sediment Transport by Waves and Currents. 2nd Edition, Delft Hydraulics, H461.

APPENDIX A: Adopted Wave Time Series



Adopted Sea and Swell Wave Time Series



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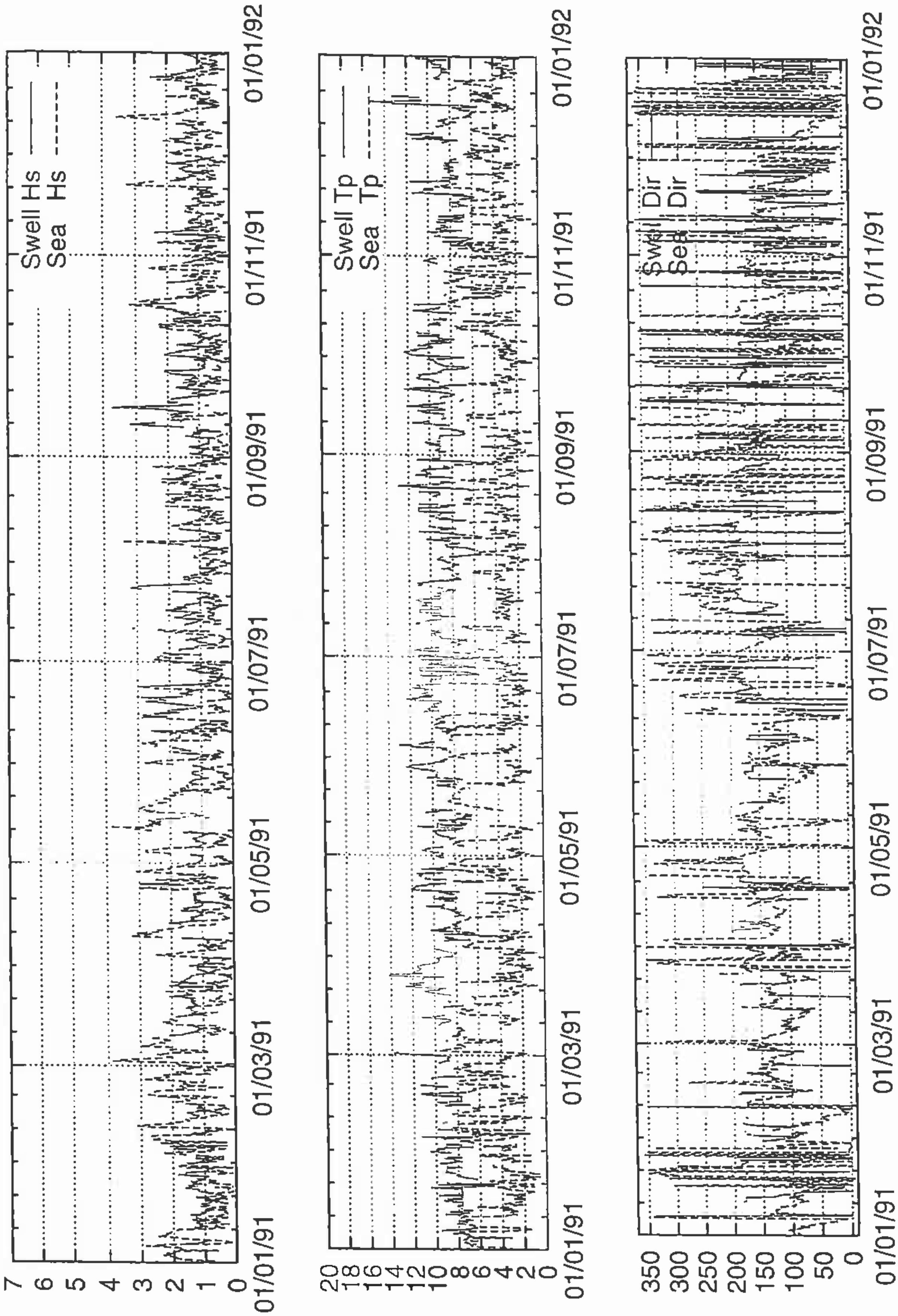

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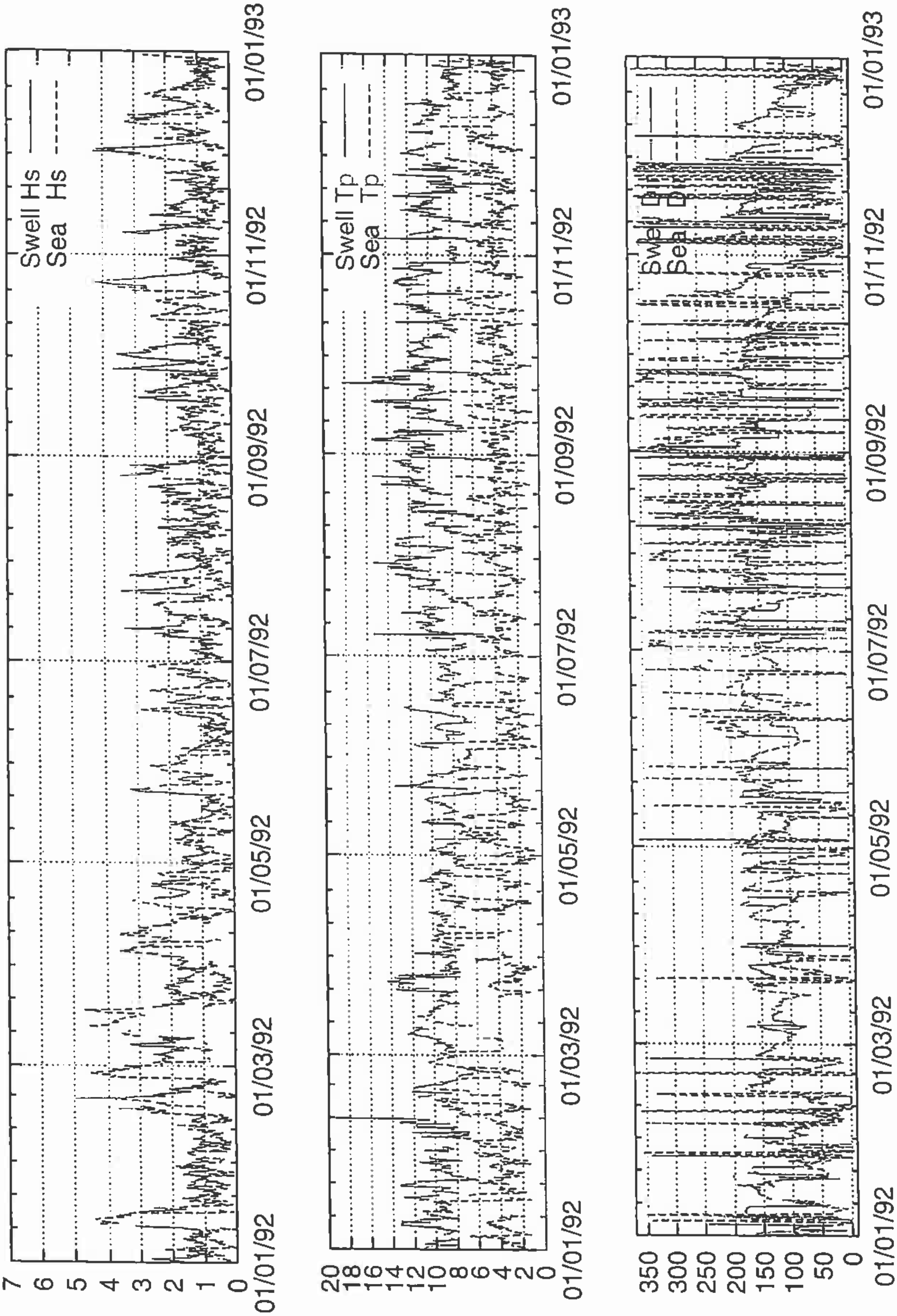

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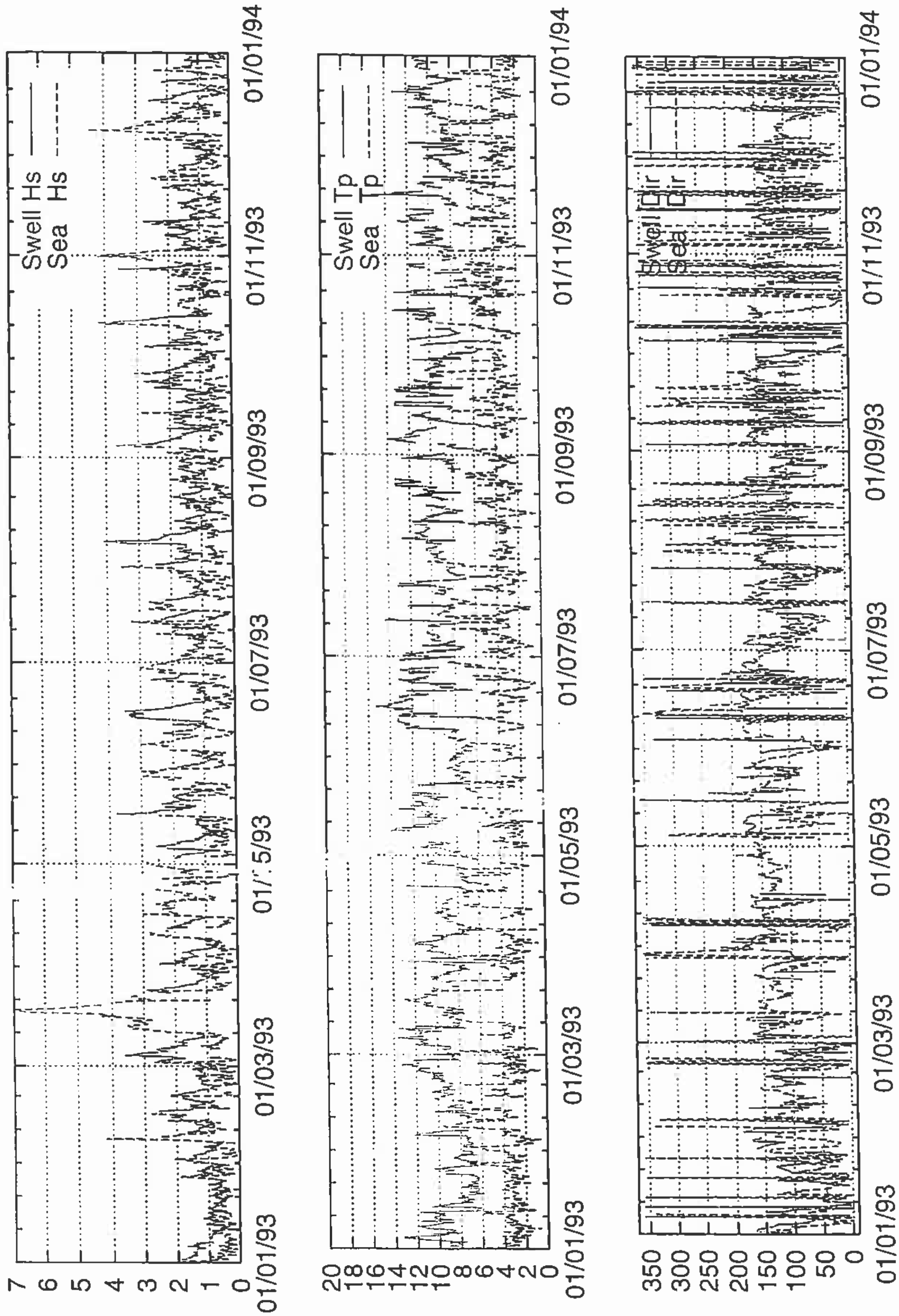

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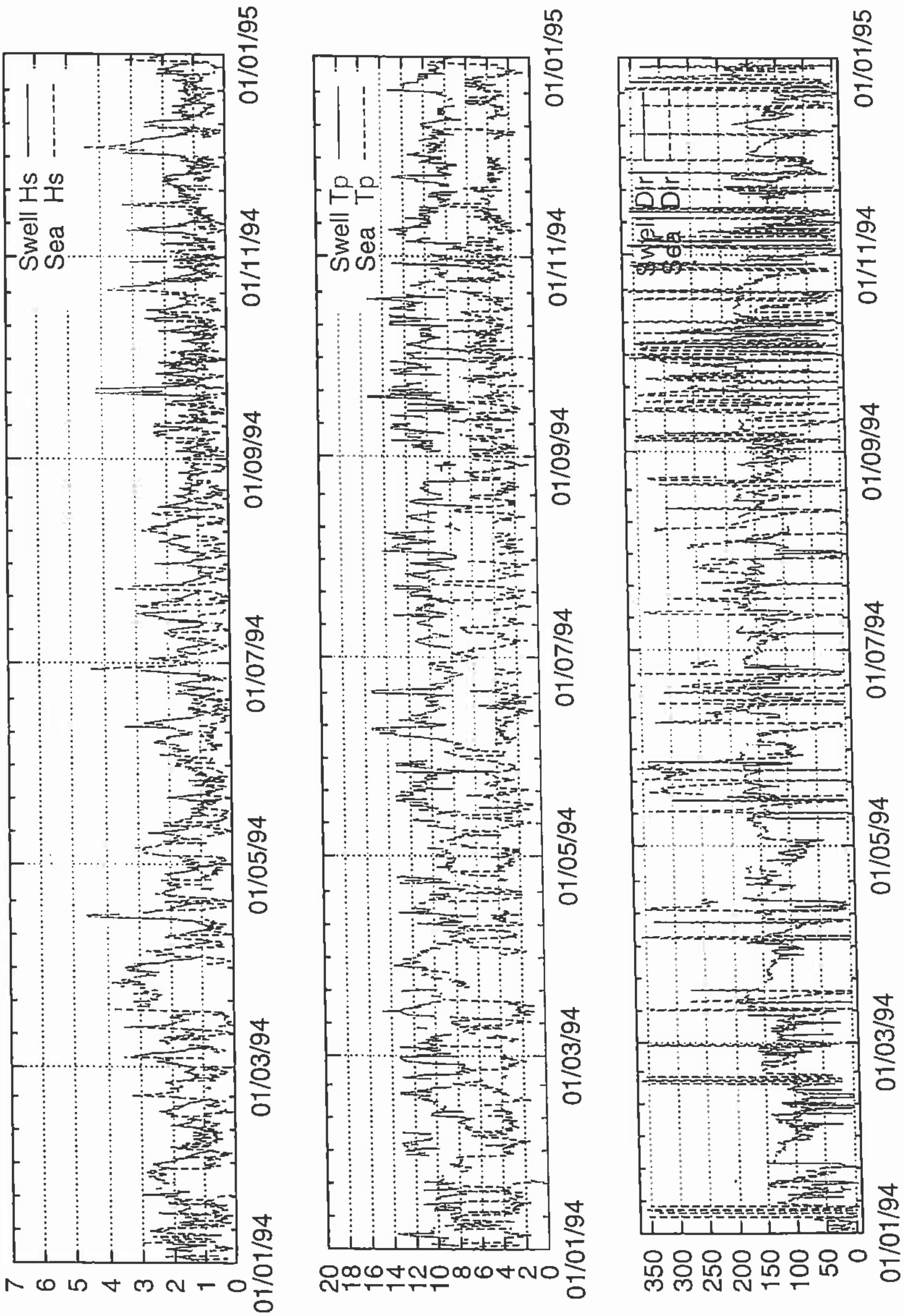

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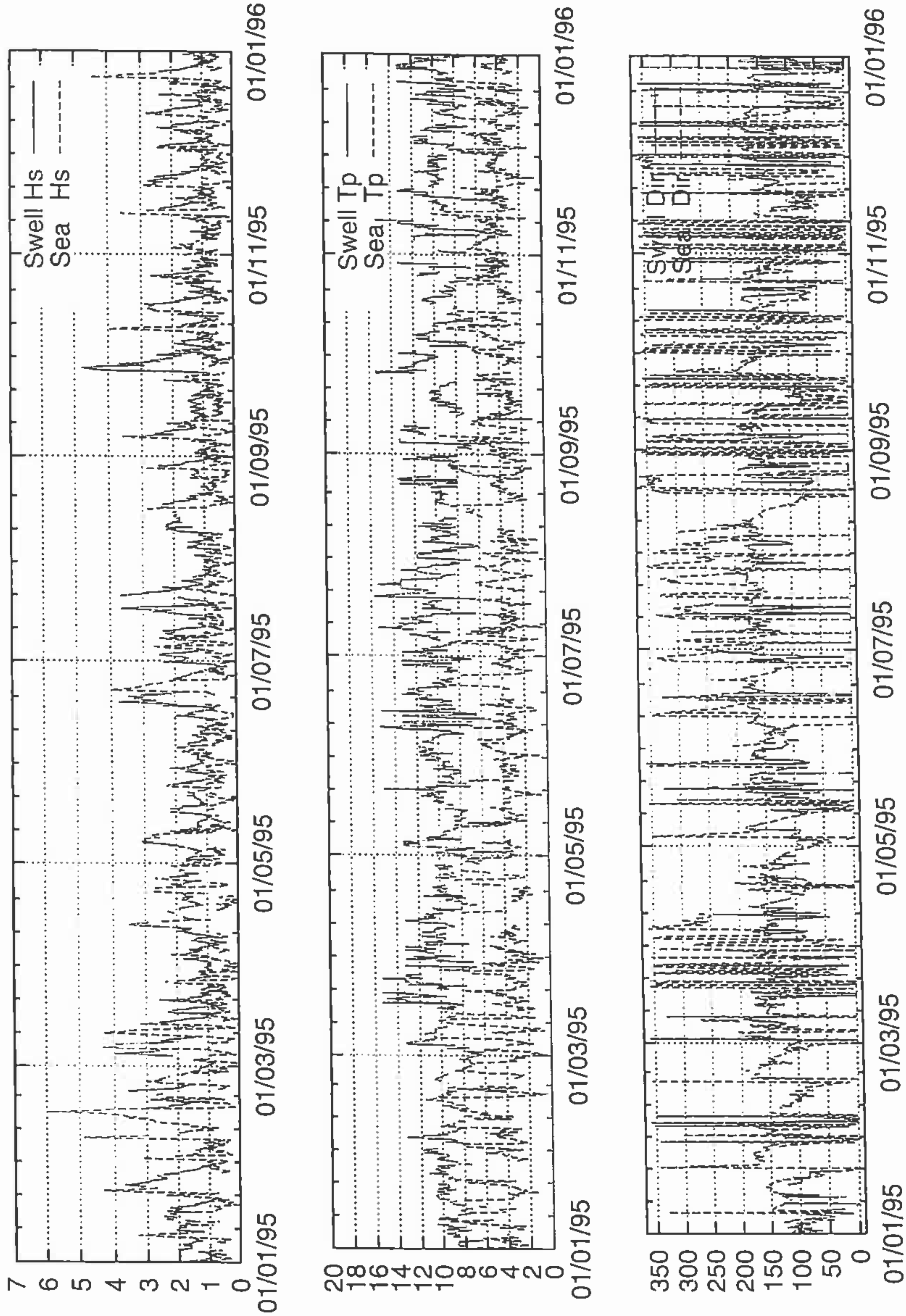

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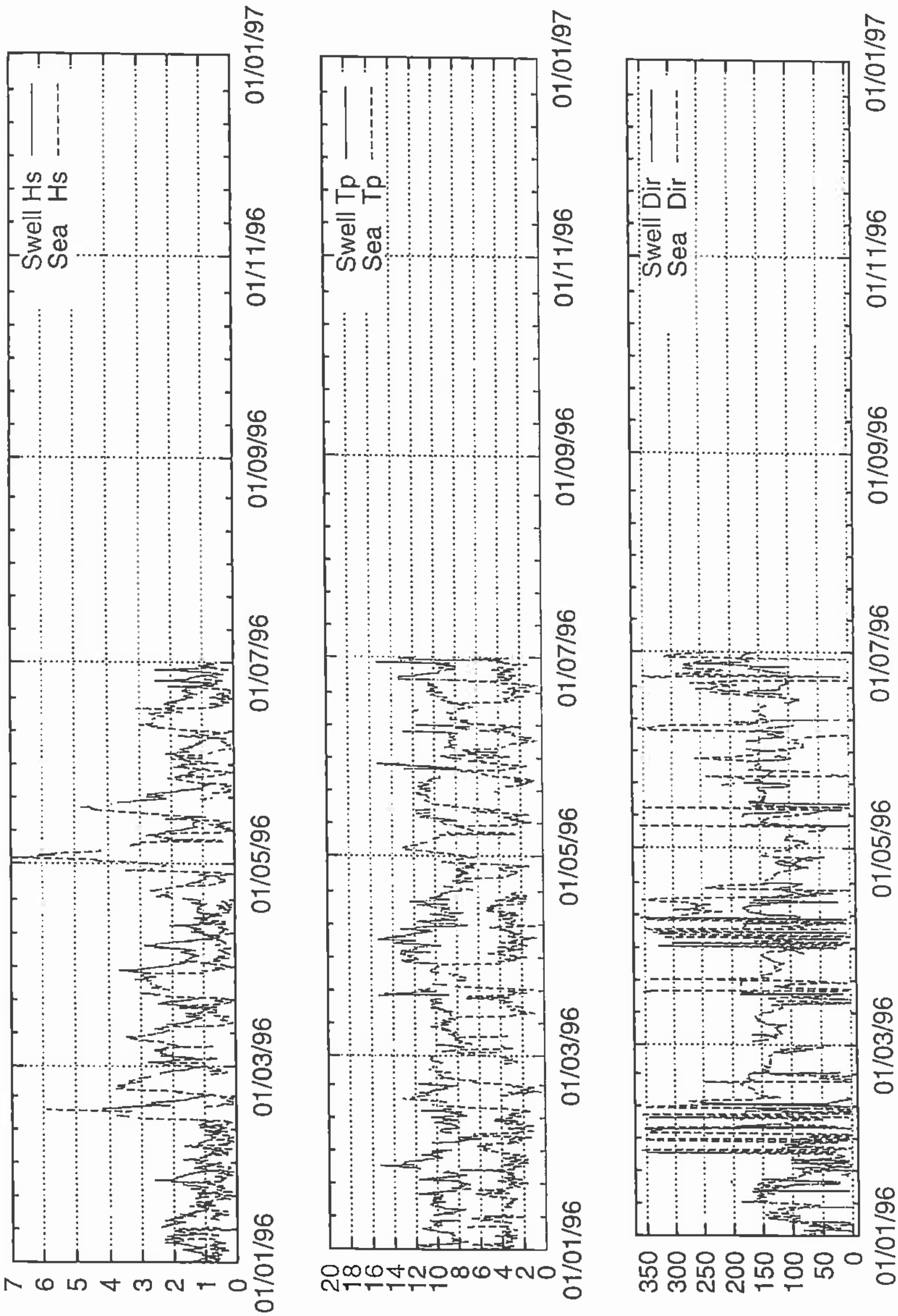
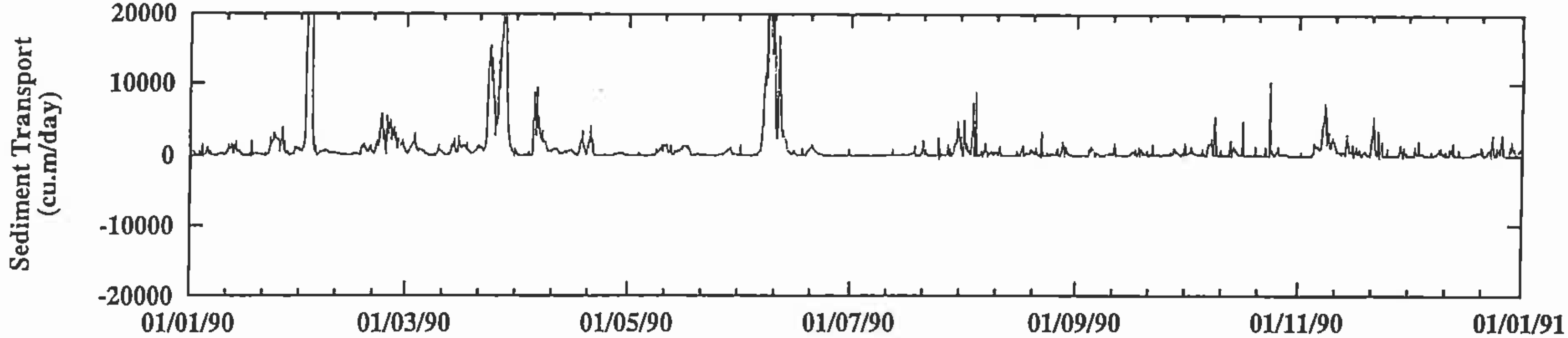
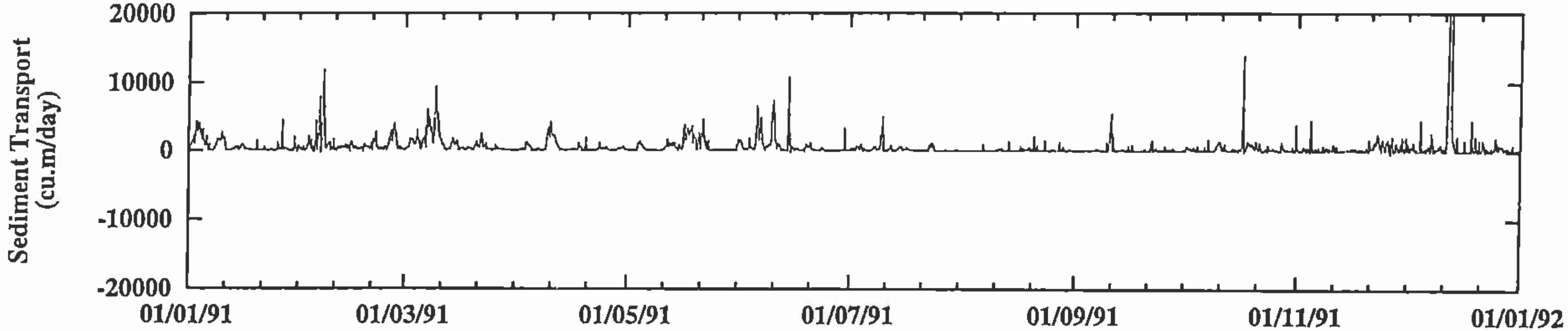


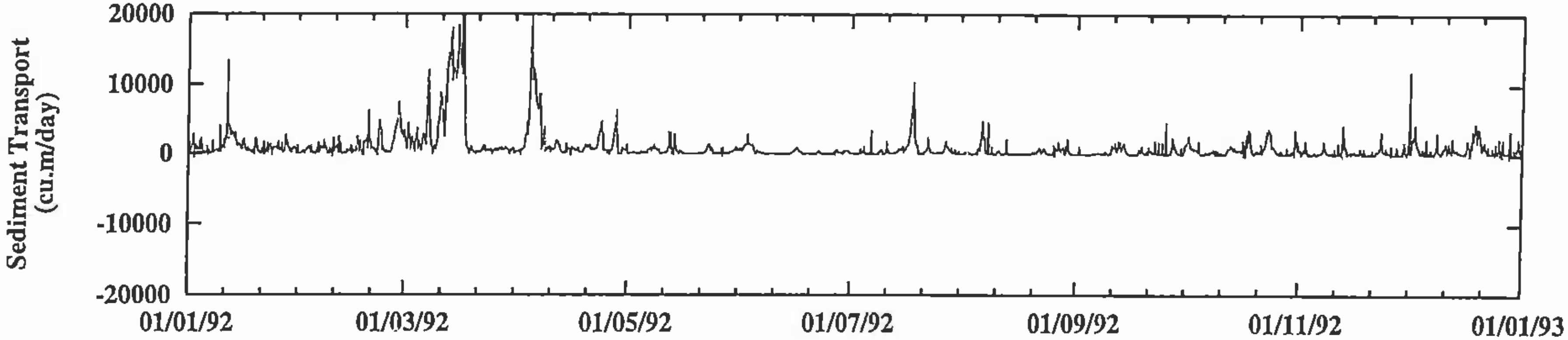
Adopted Sea and Swell Wave Time Series

APPENDIX B: Longshore Transport Time Series

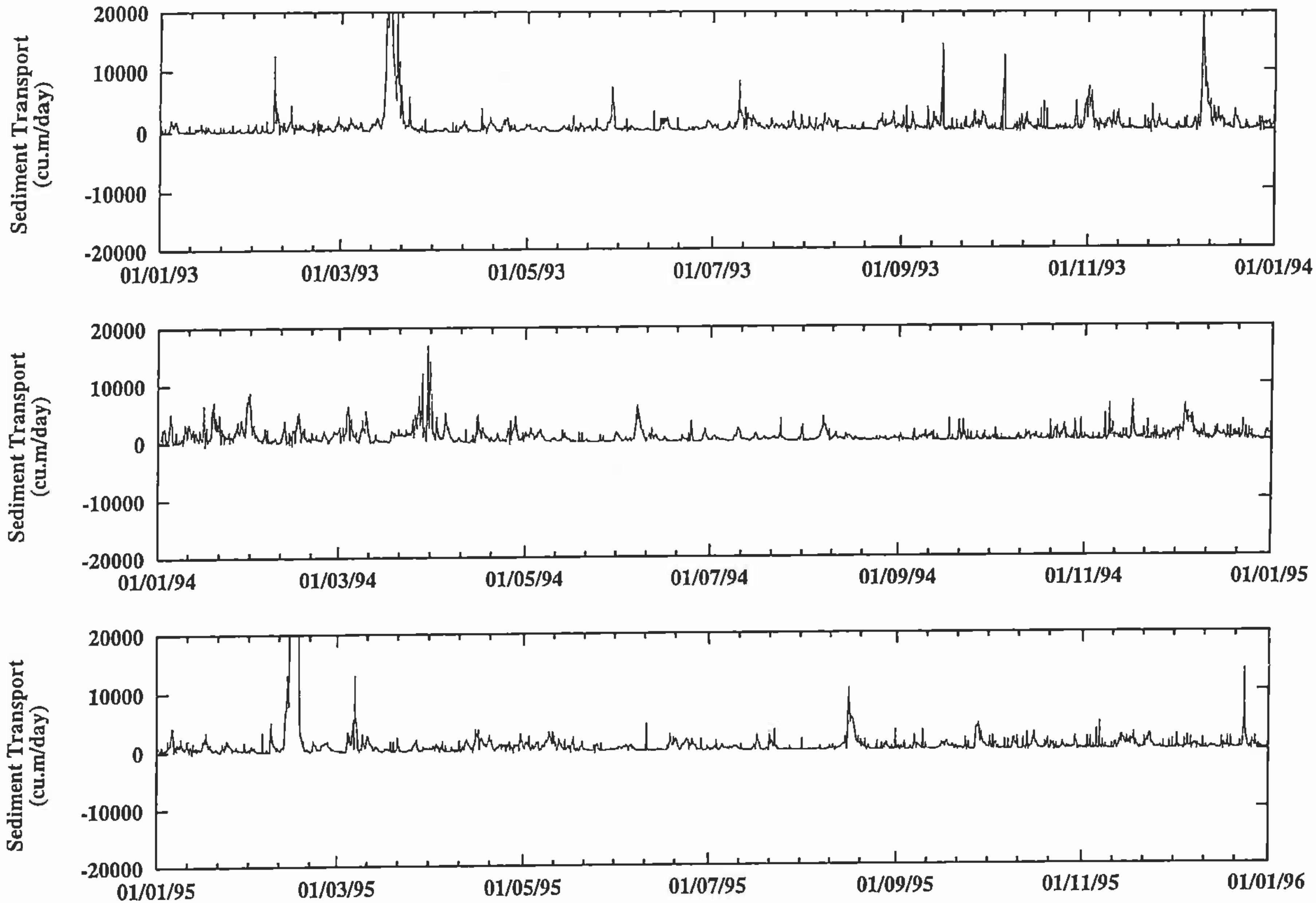
Daily Longshore Sand Transport
North Kirra



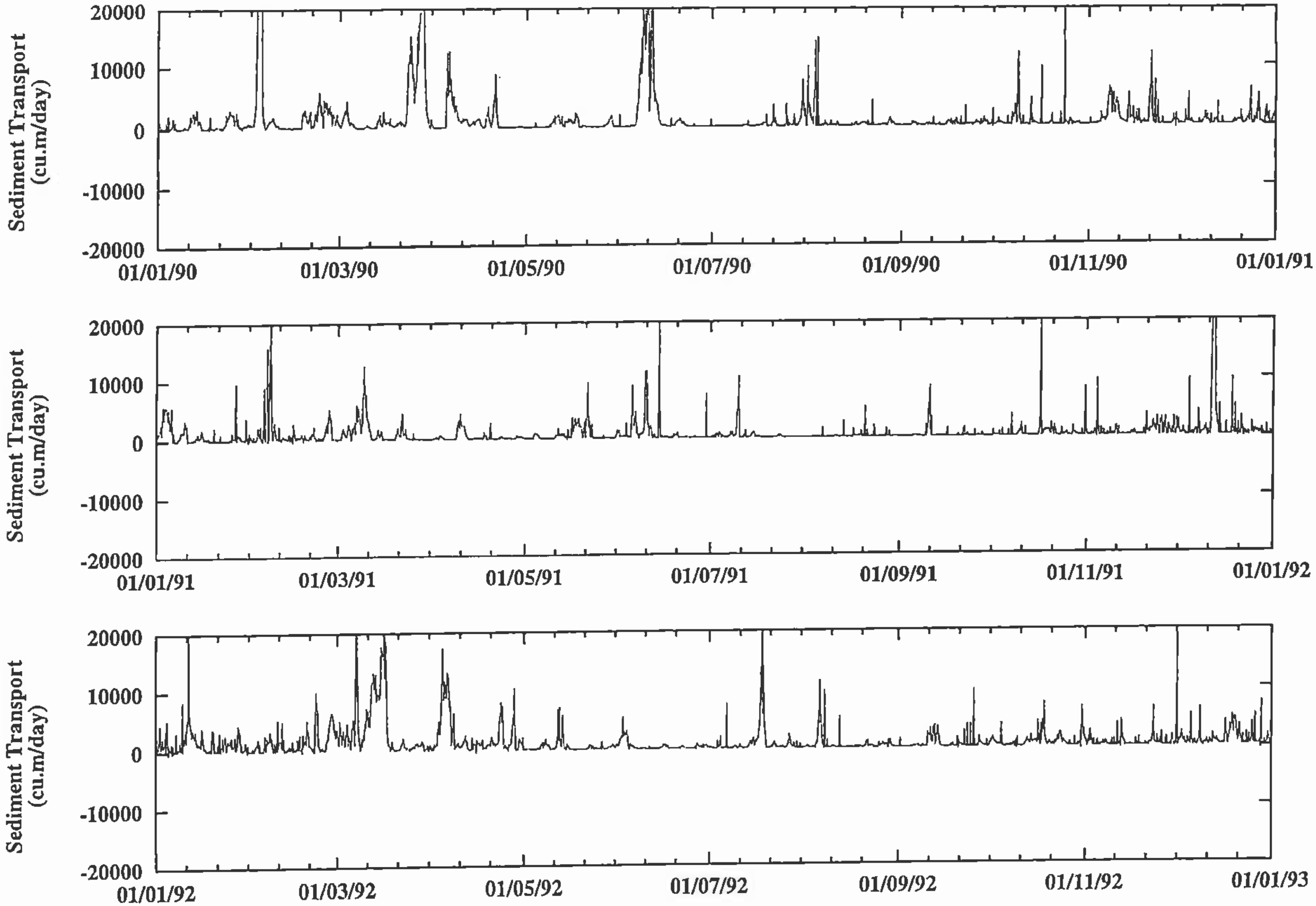


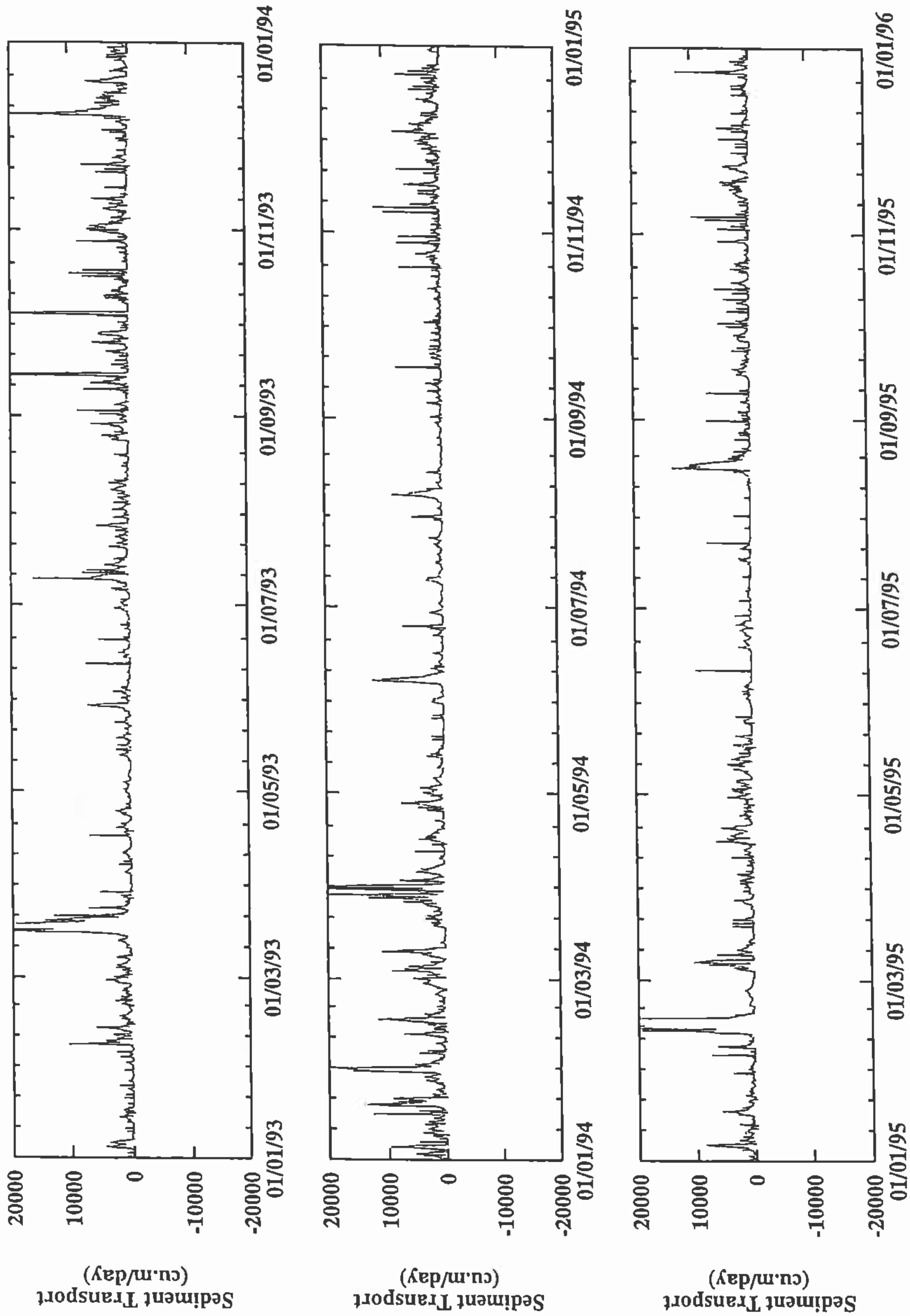


Daily Longshore Sand Transport
North Kirra



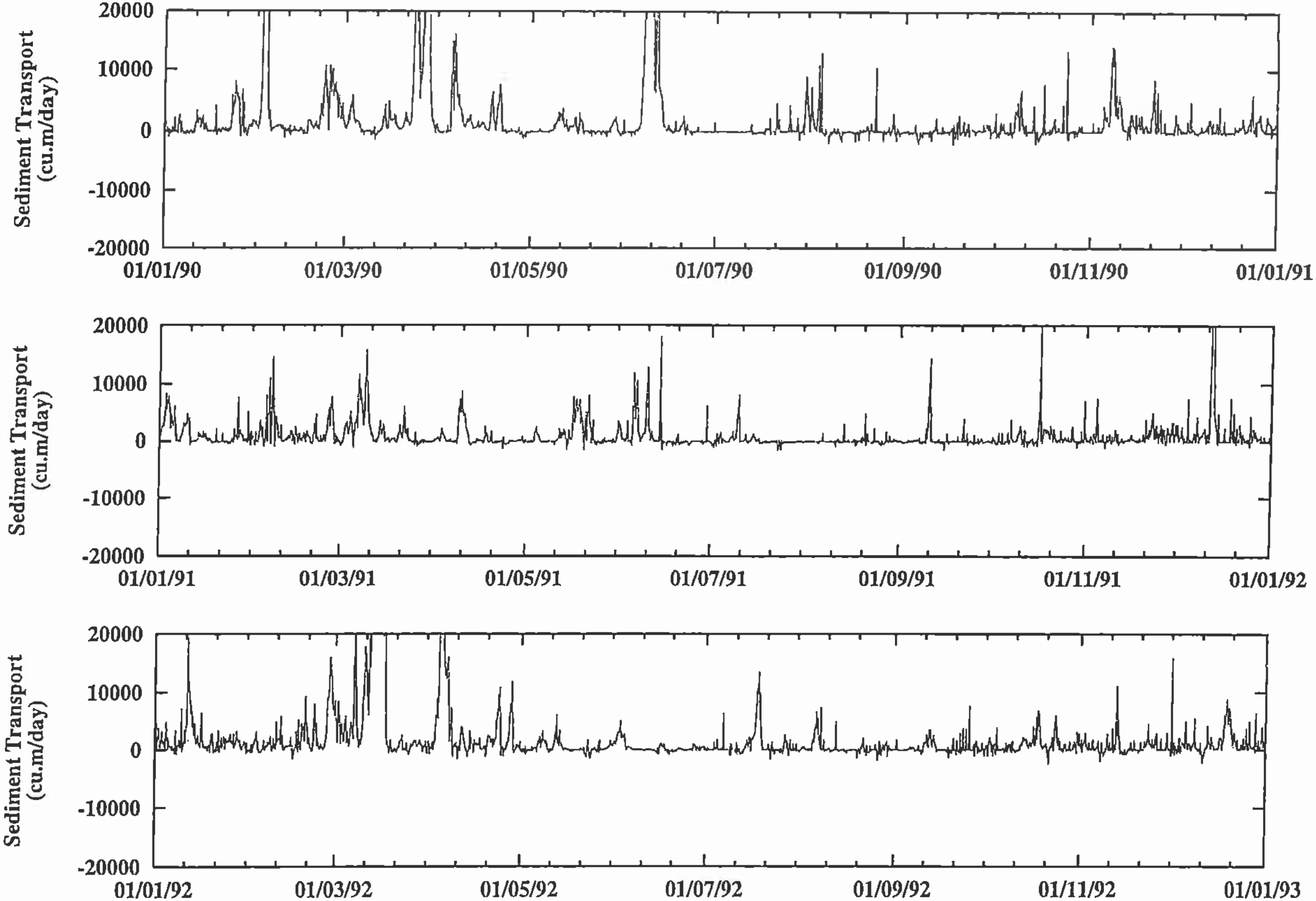
Daily Longshore Sand Transport
Kirra



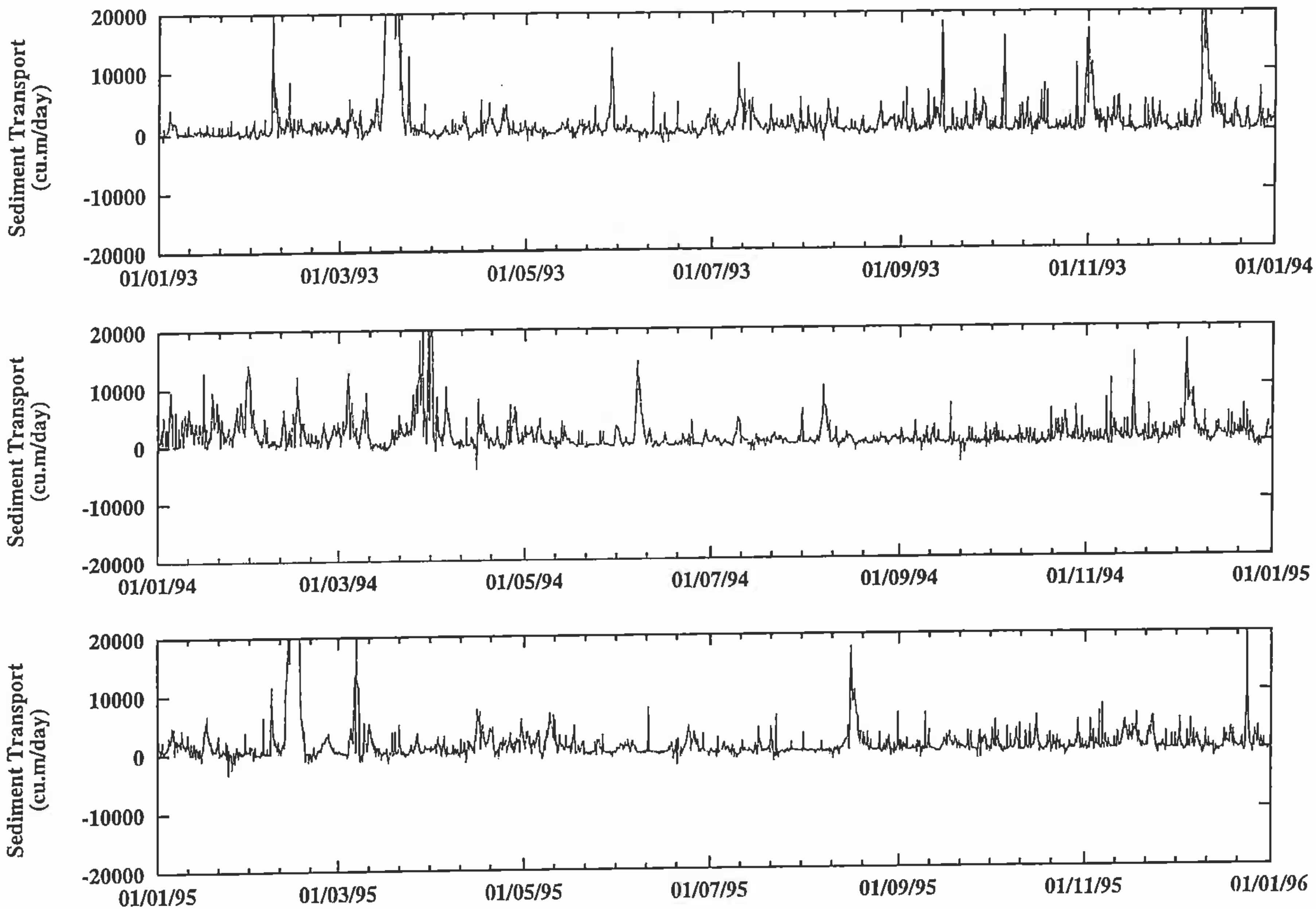


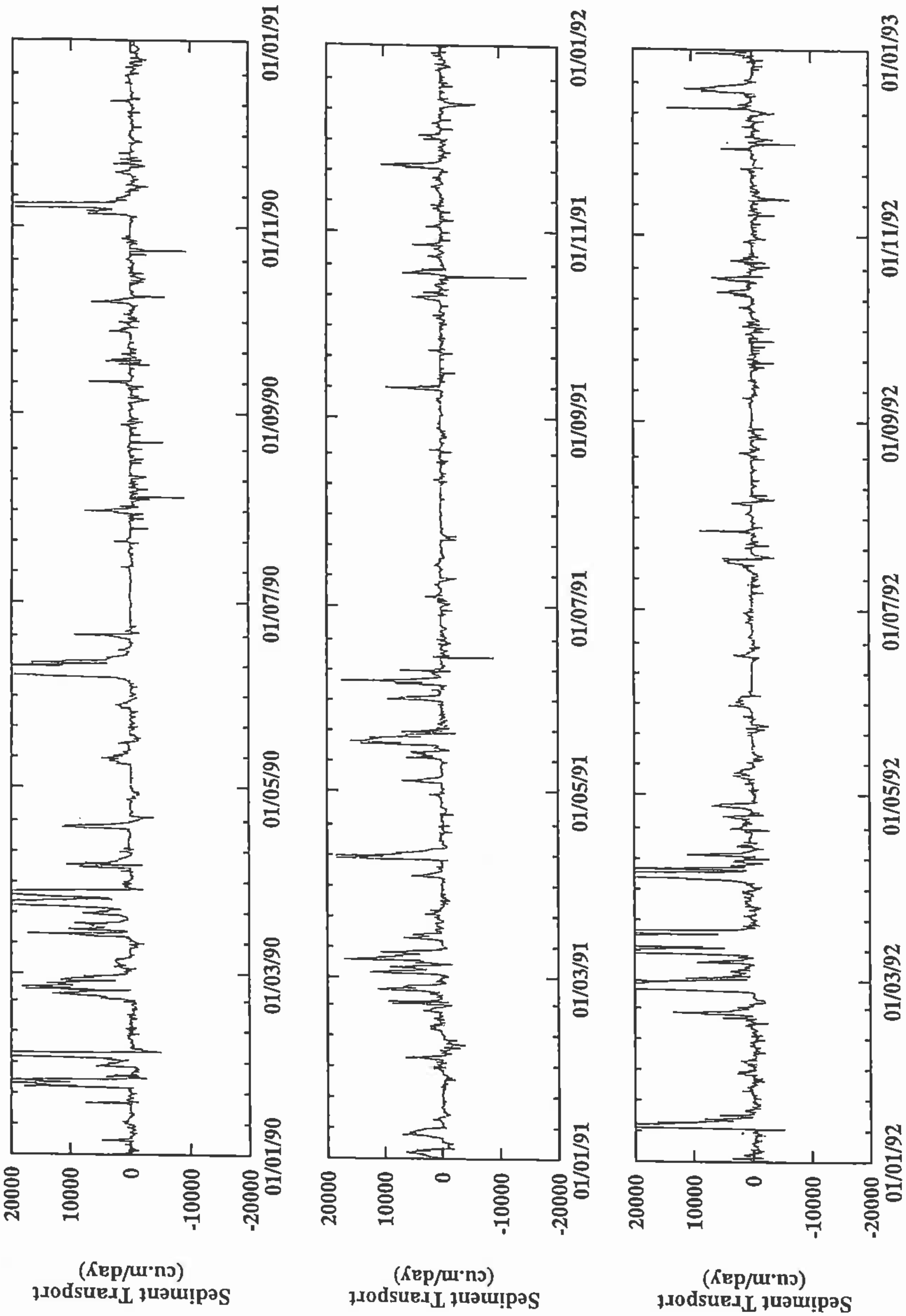
Daily Longshore Sand Transport
Kirra

Daily Longshore Sand Transport
Snapper Rocks



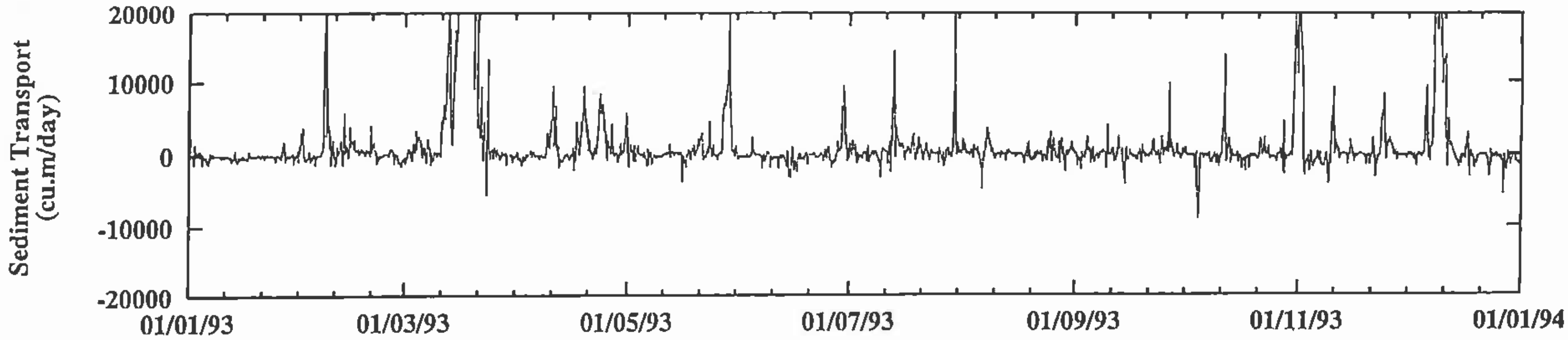
Daily Longshore Sand Transport
Snapper Rocks

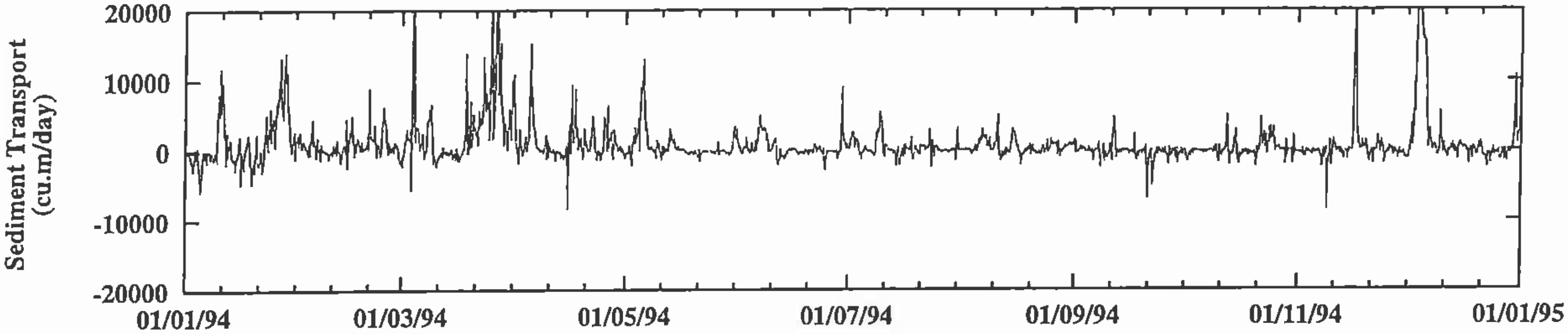


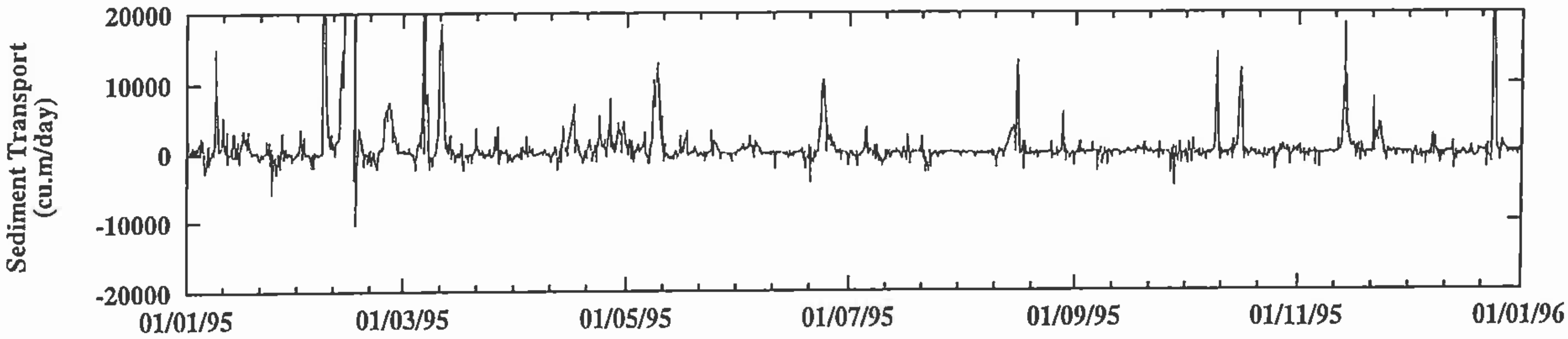


**Daily Longshore Sand Transport
Point Danger (Frog's Beach)**

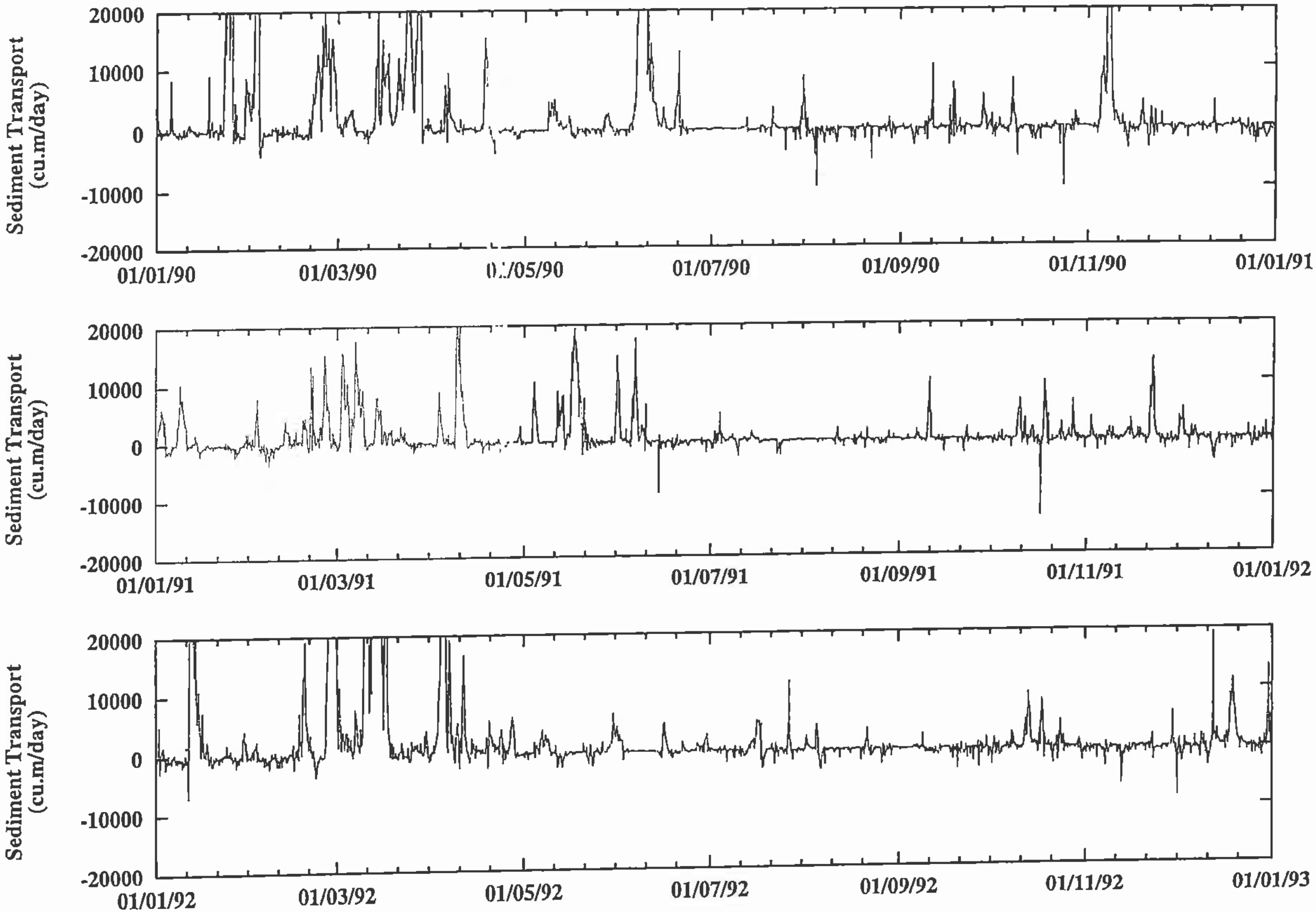
Daily Longshore Sand Transport
Point Danger (Frog's Beach)

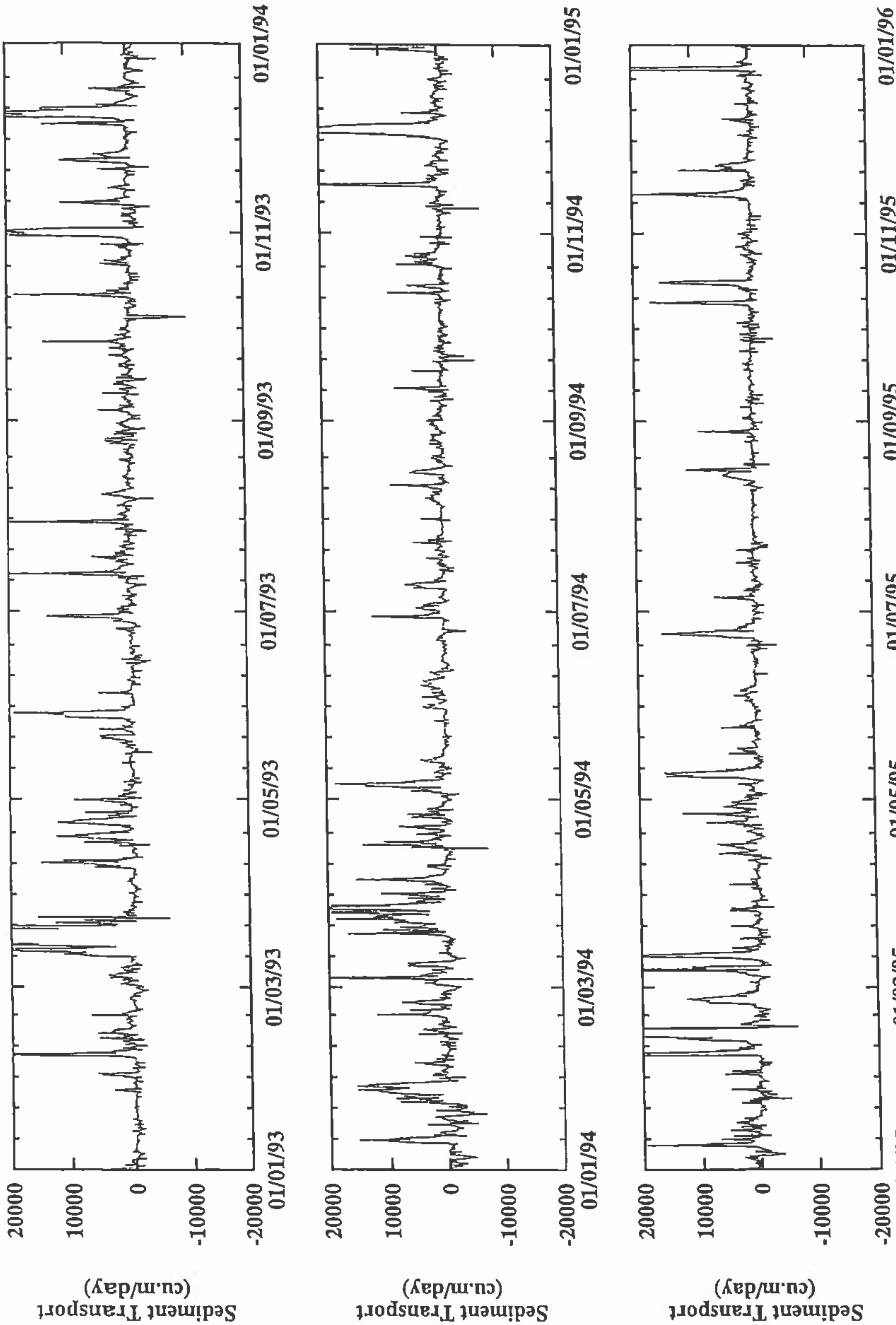






Daily Longshore Sand Transport
Lovers Rock





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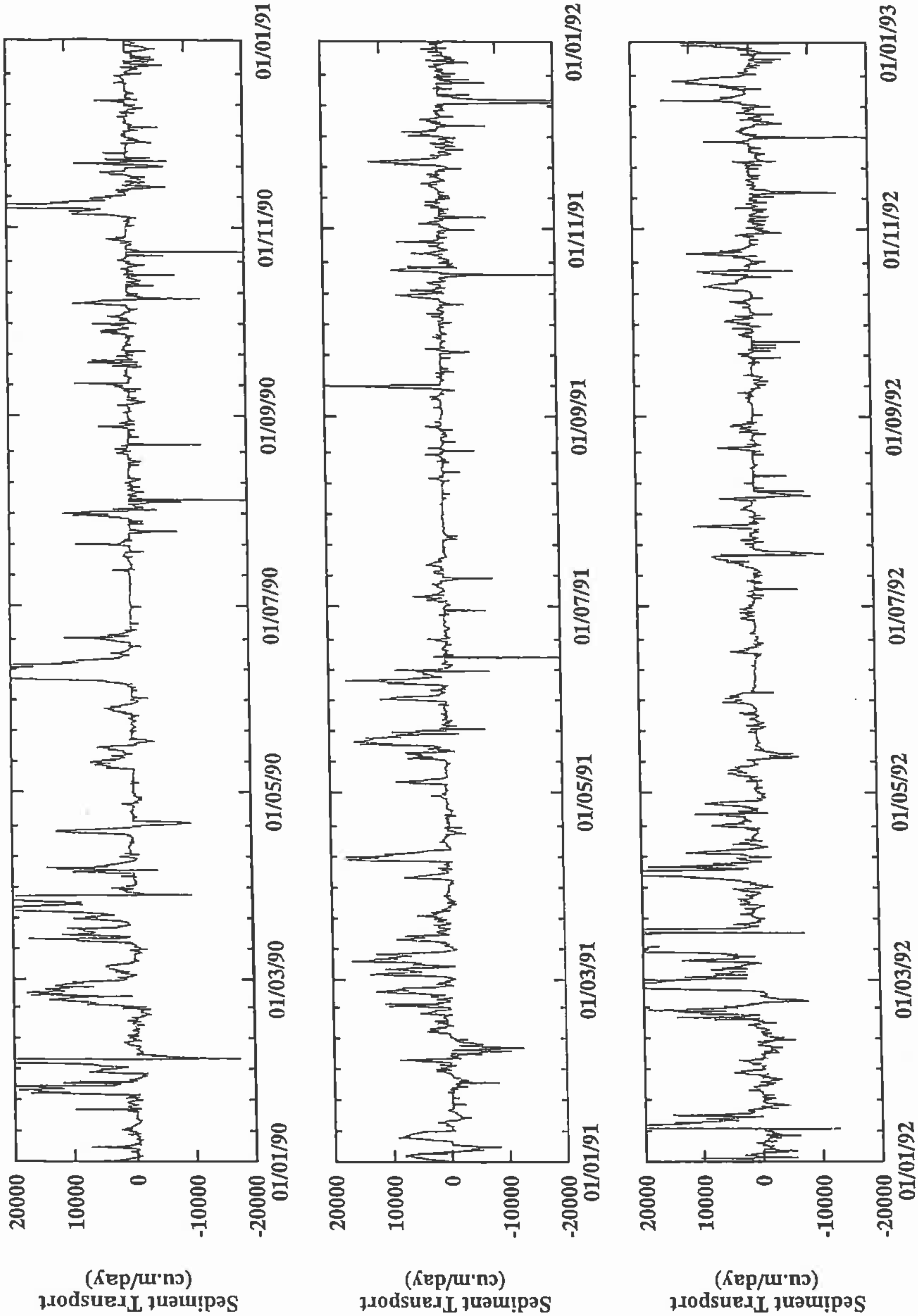
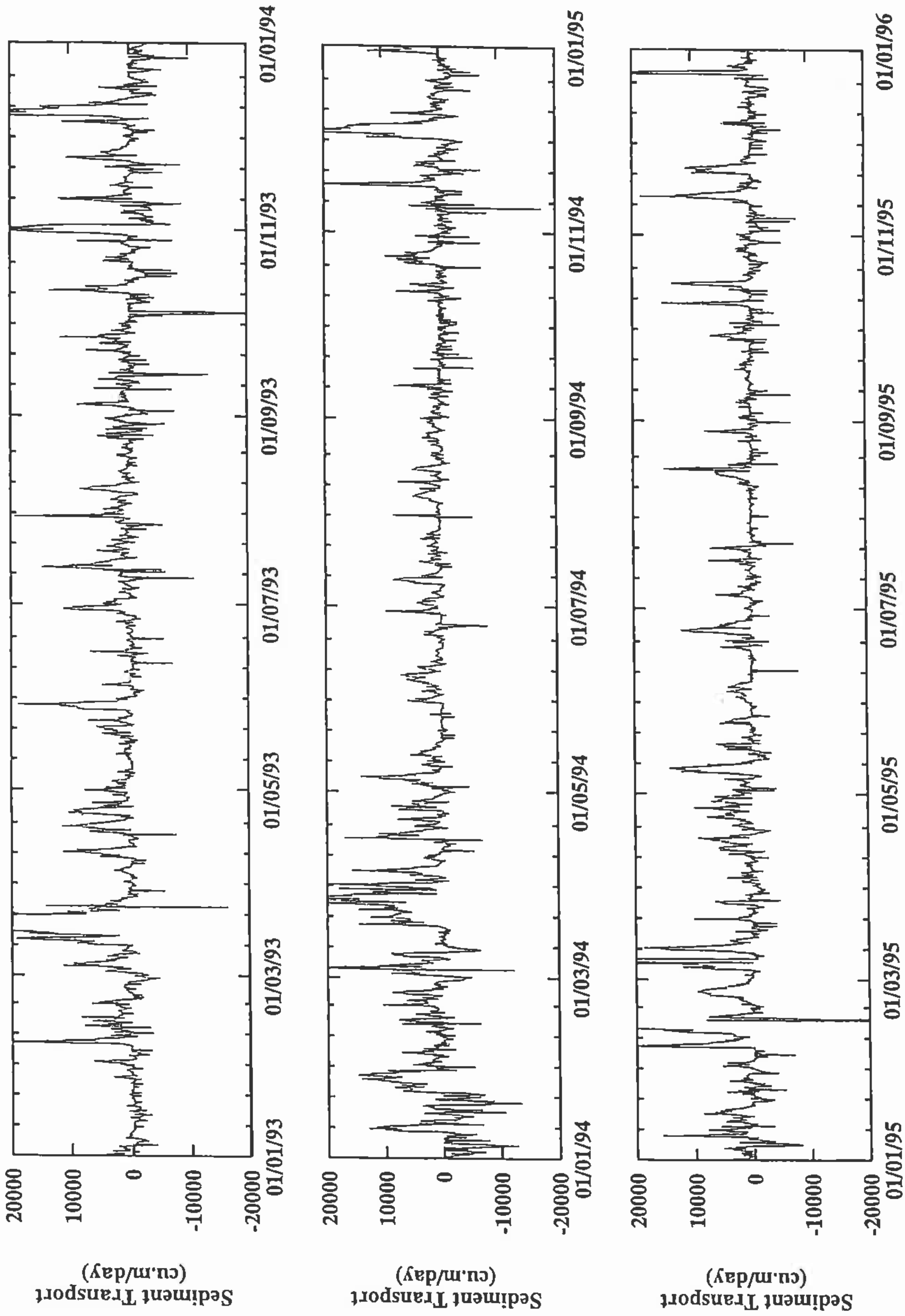
Lovers Rock



Daily Longshore Sand Transport
Duranbah



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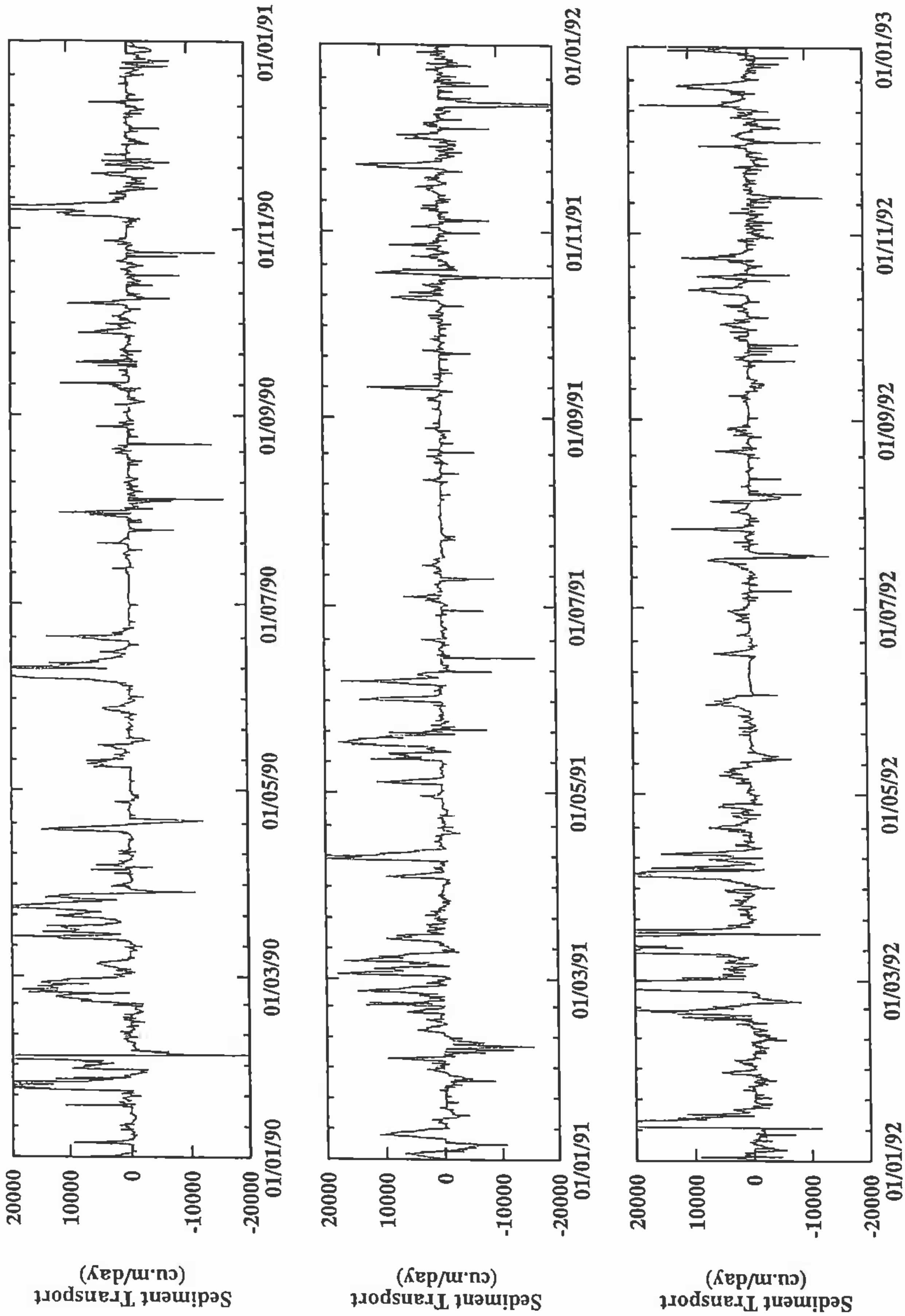
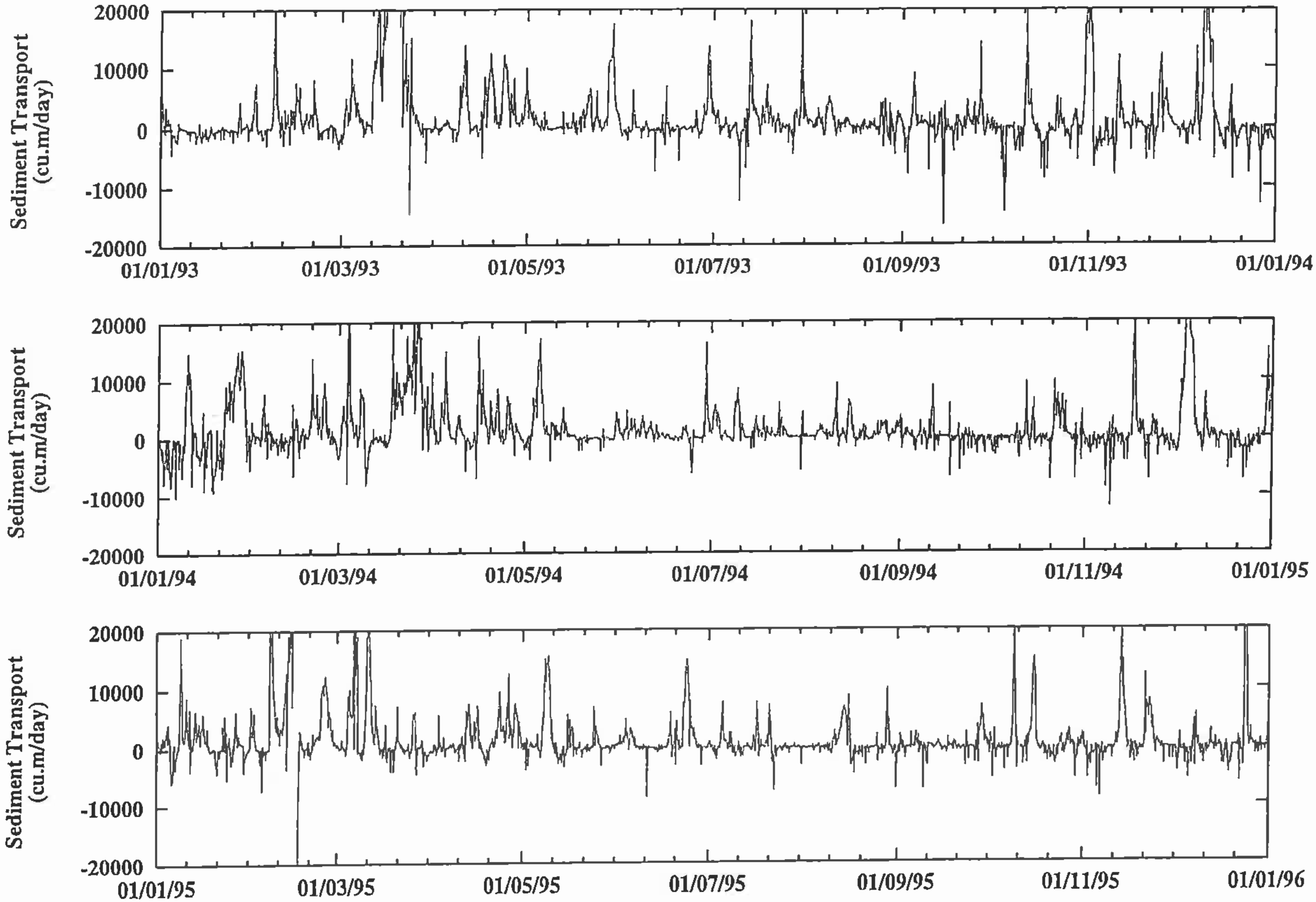
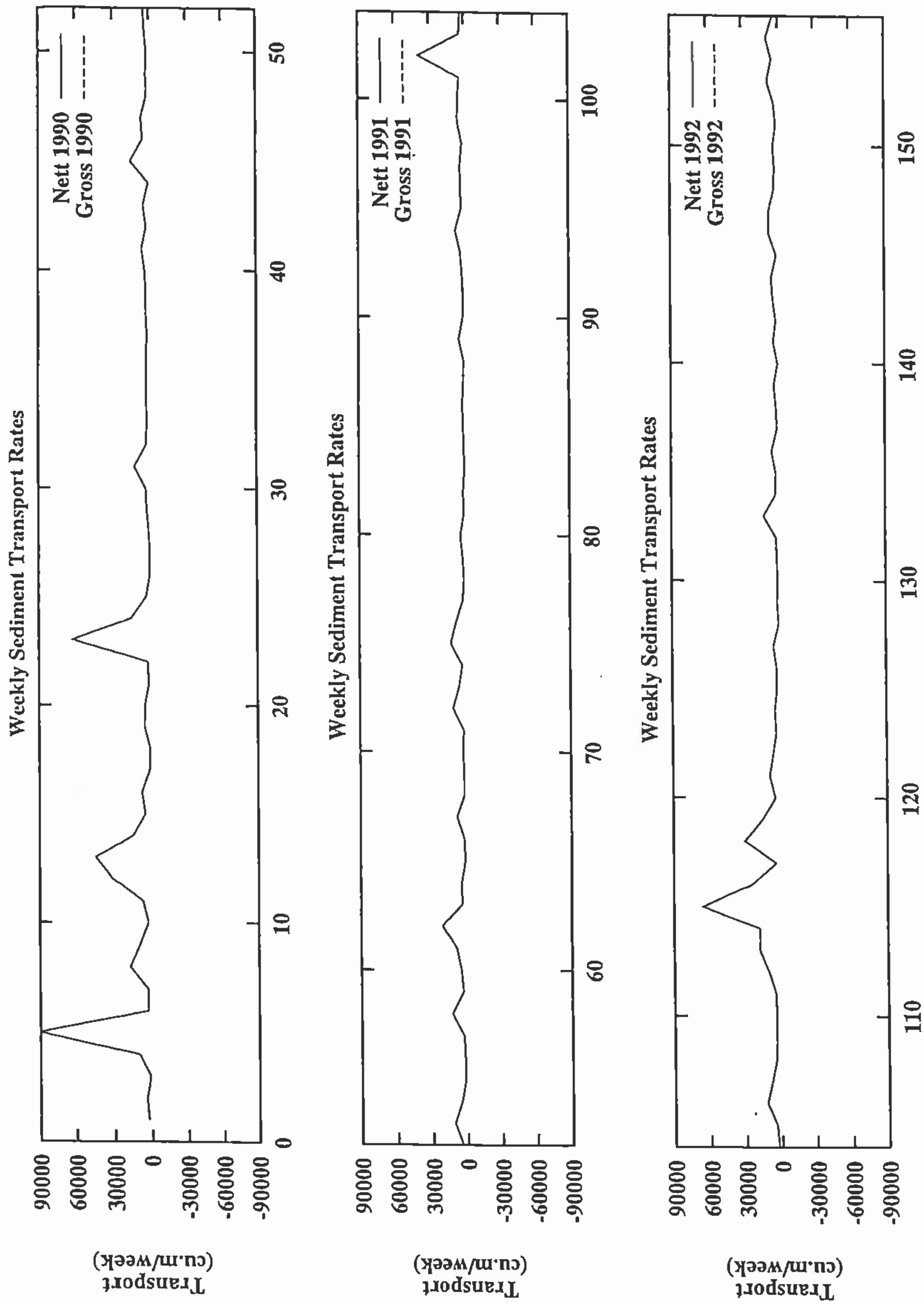
Duranbah



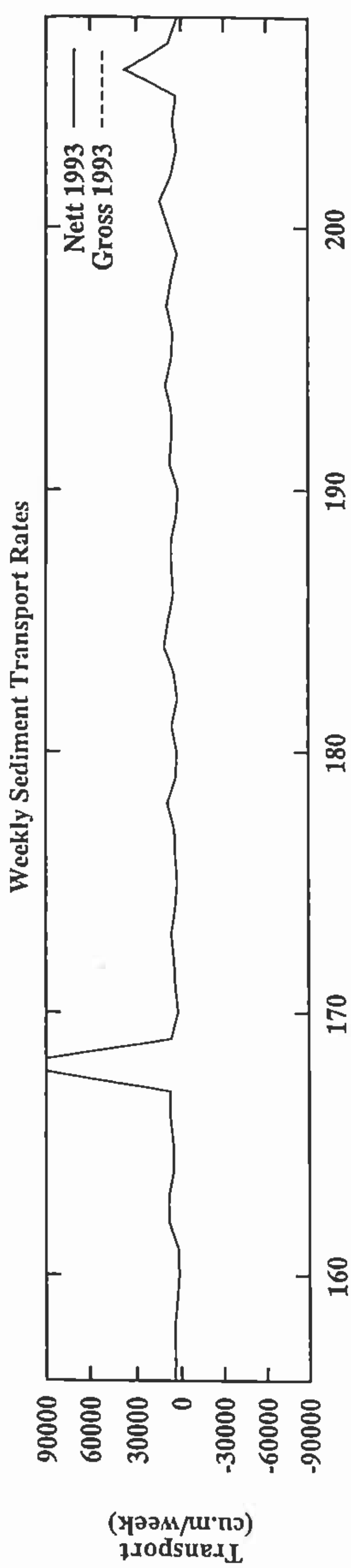
Daily Longshore Sand Transport
Letitia Spit

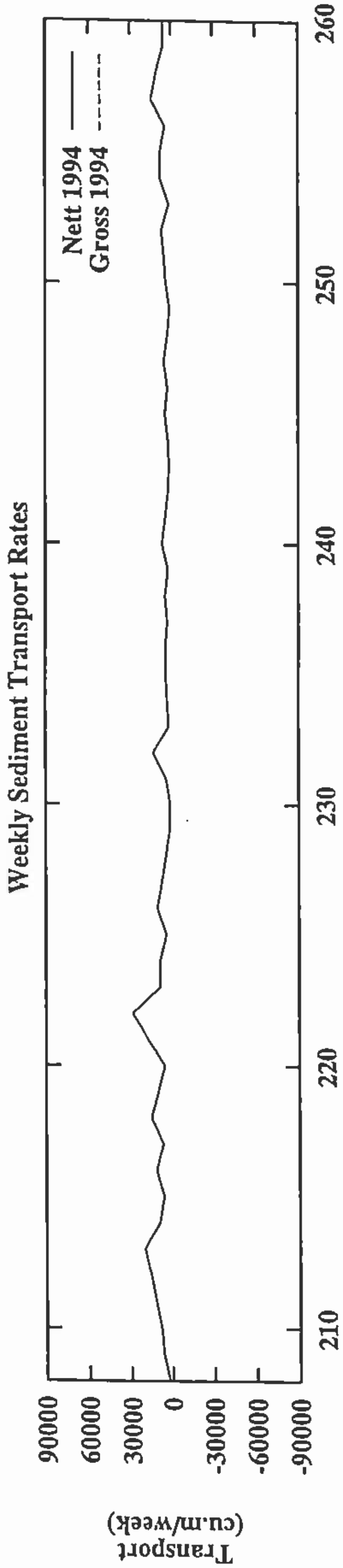
Daily Longshore Sand Transport
Letitia Spit

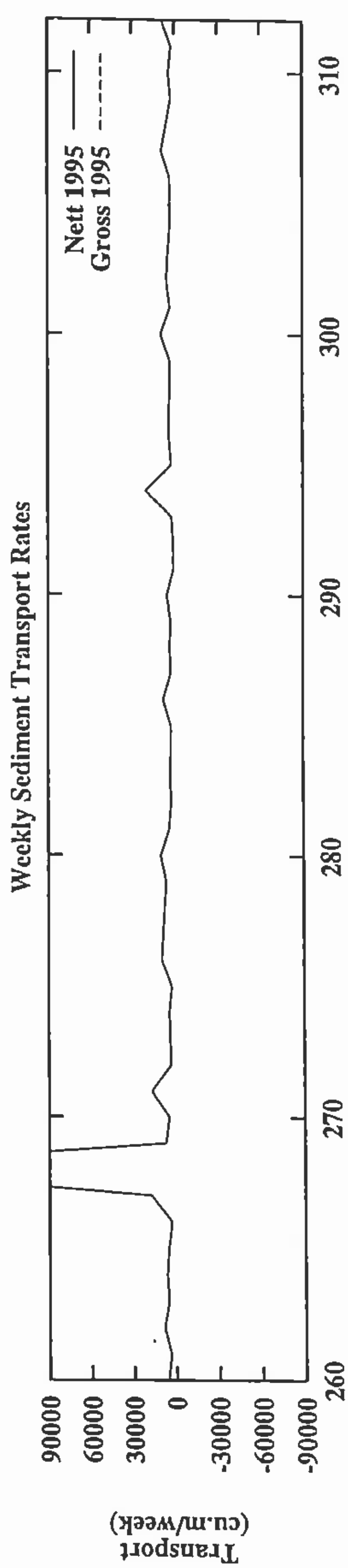




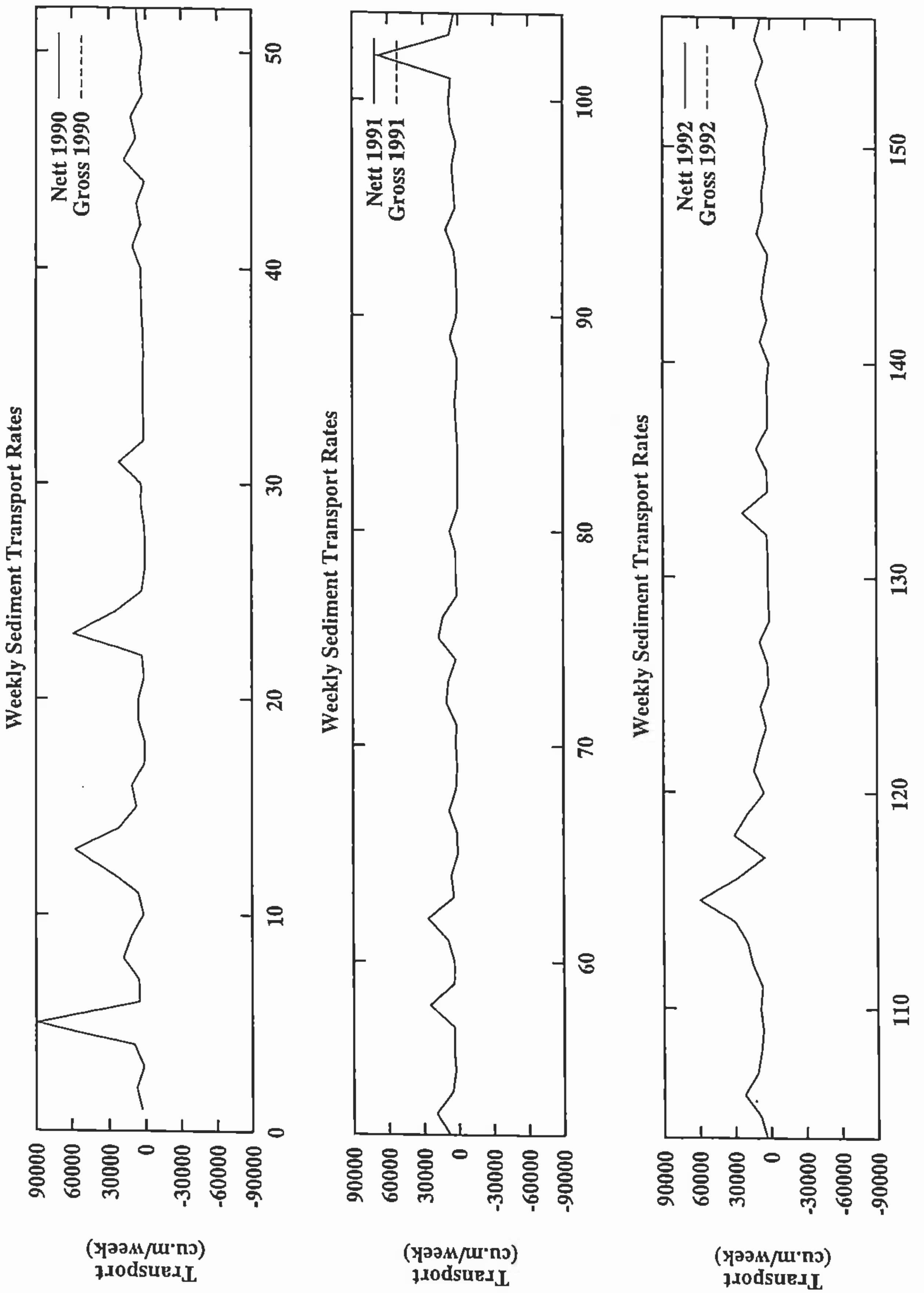
Weekly Longshore Sand Transport
North Kirra



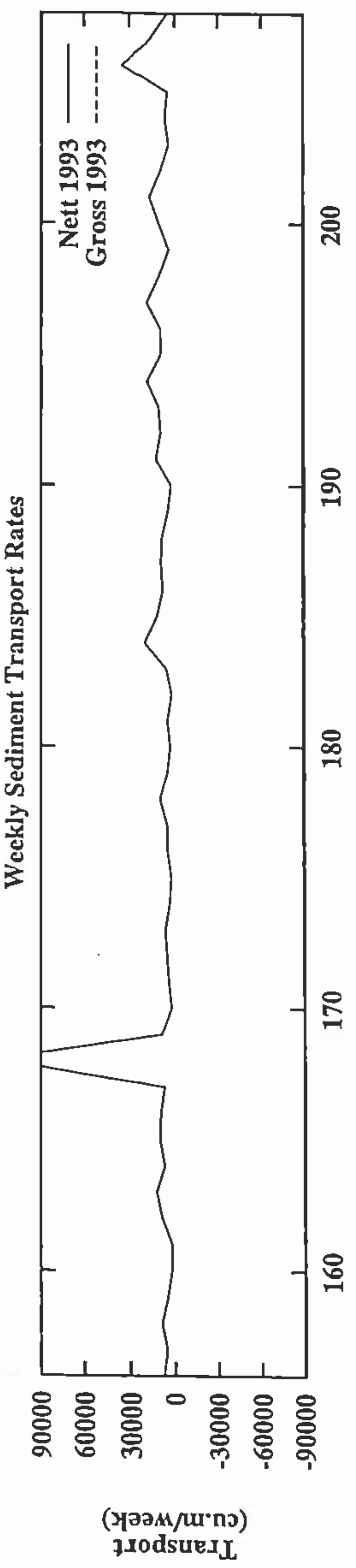


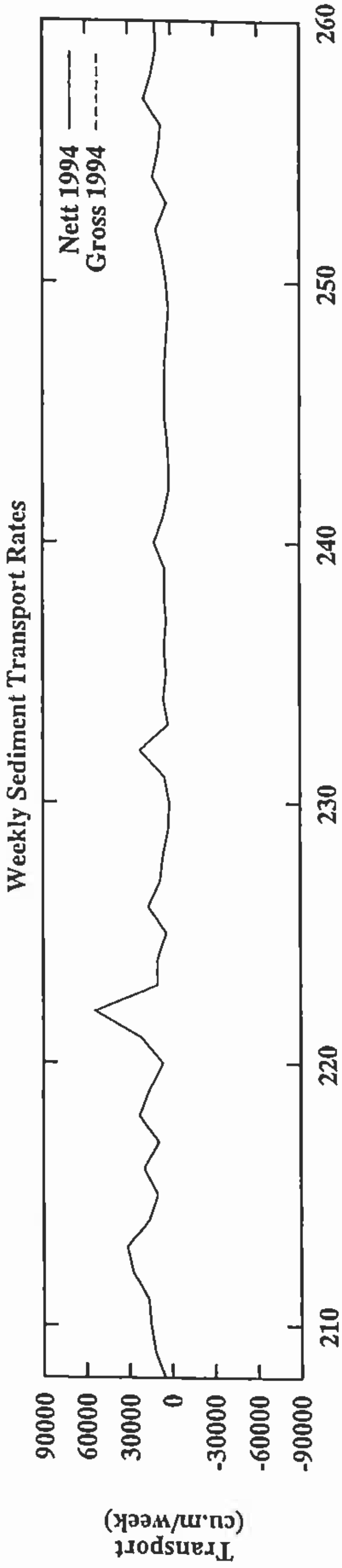


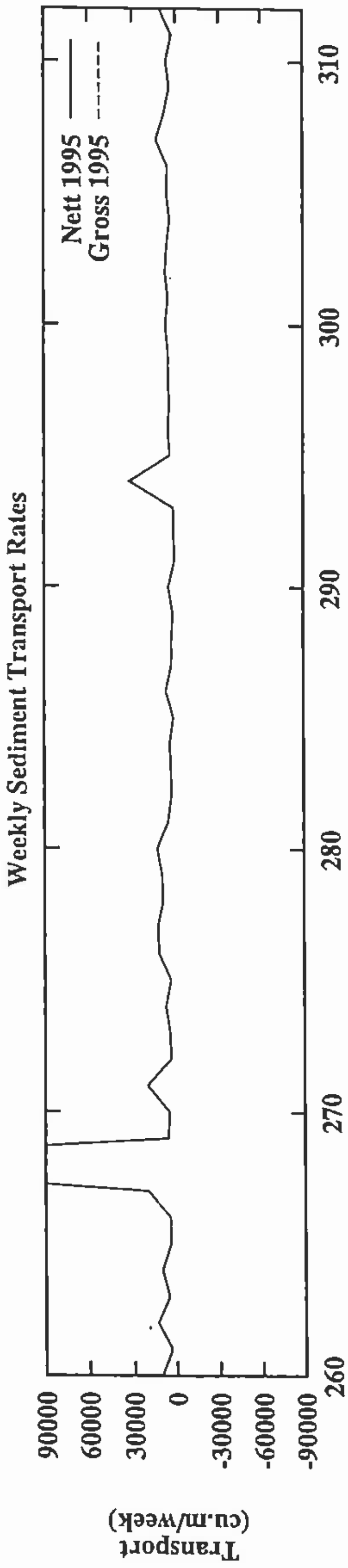
Weekly Longshore Sand Transport North Kirra



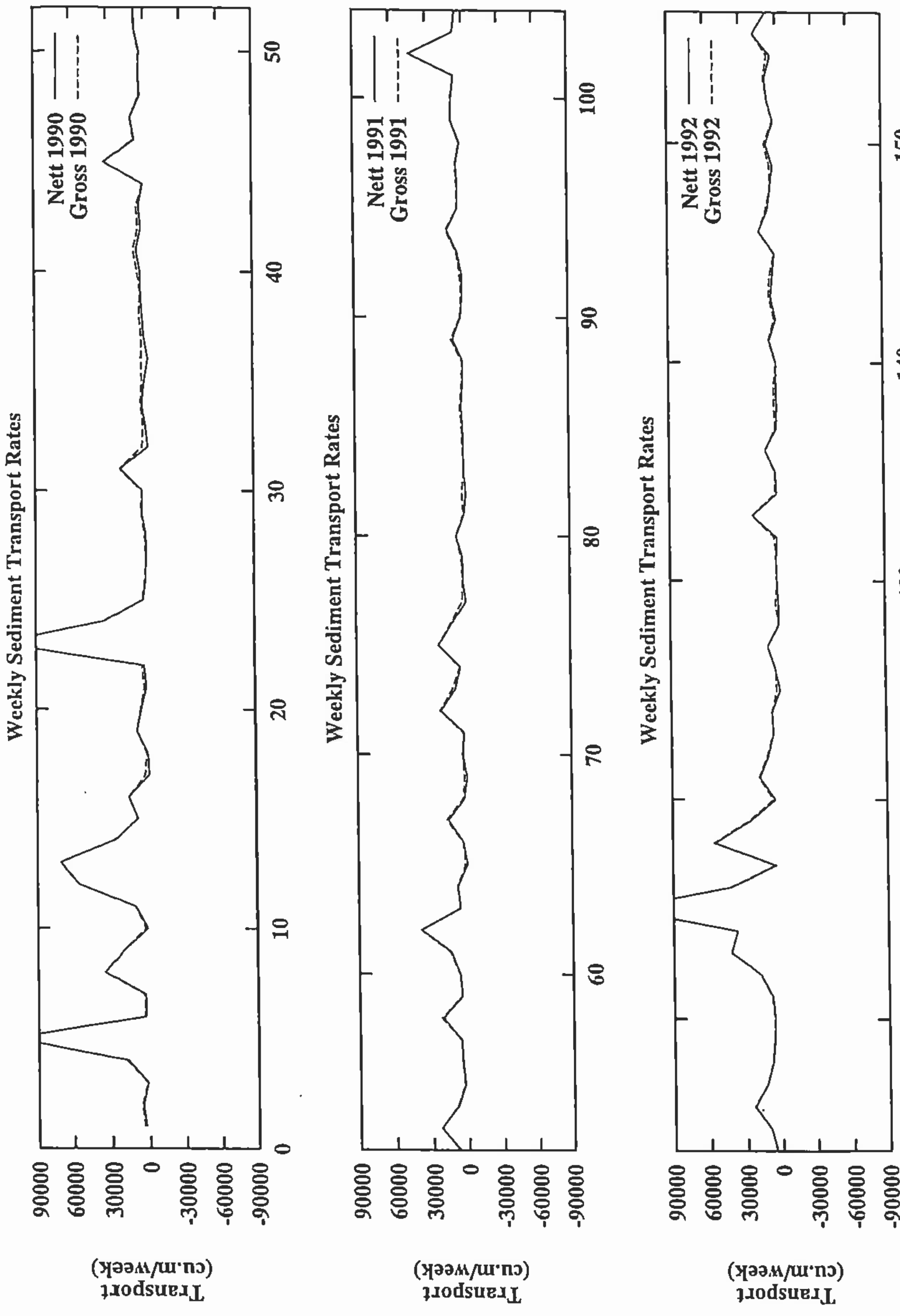
Weekly Longshore Sand Transport
Kirra



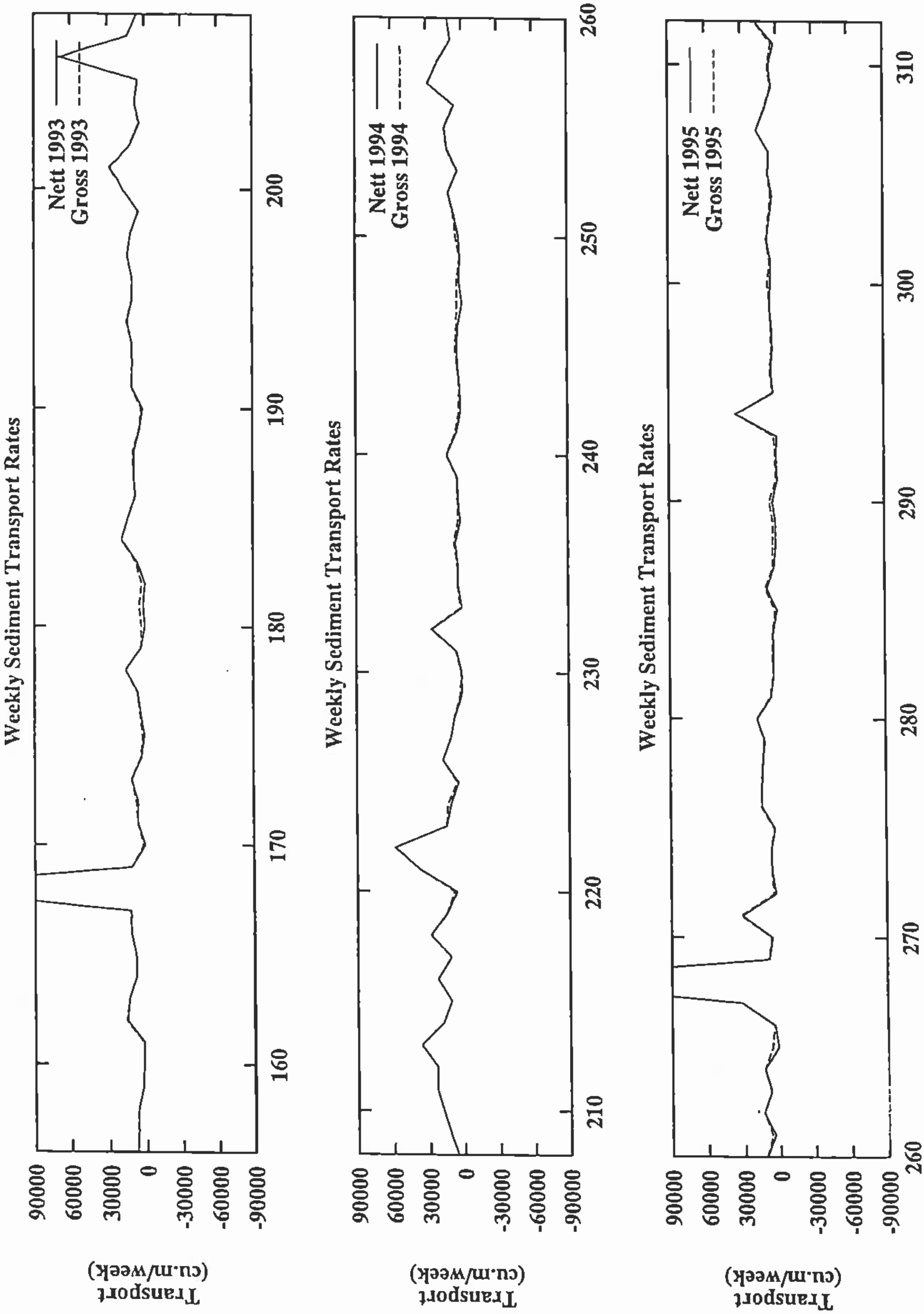




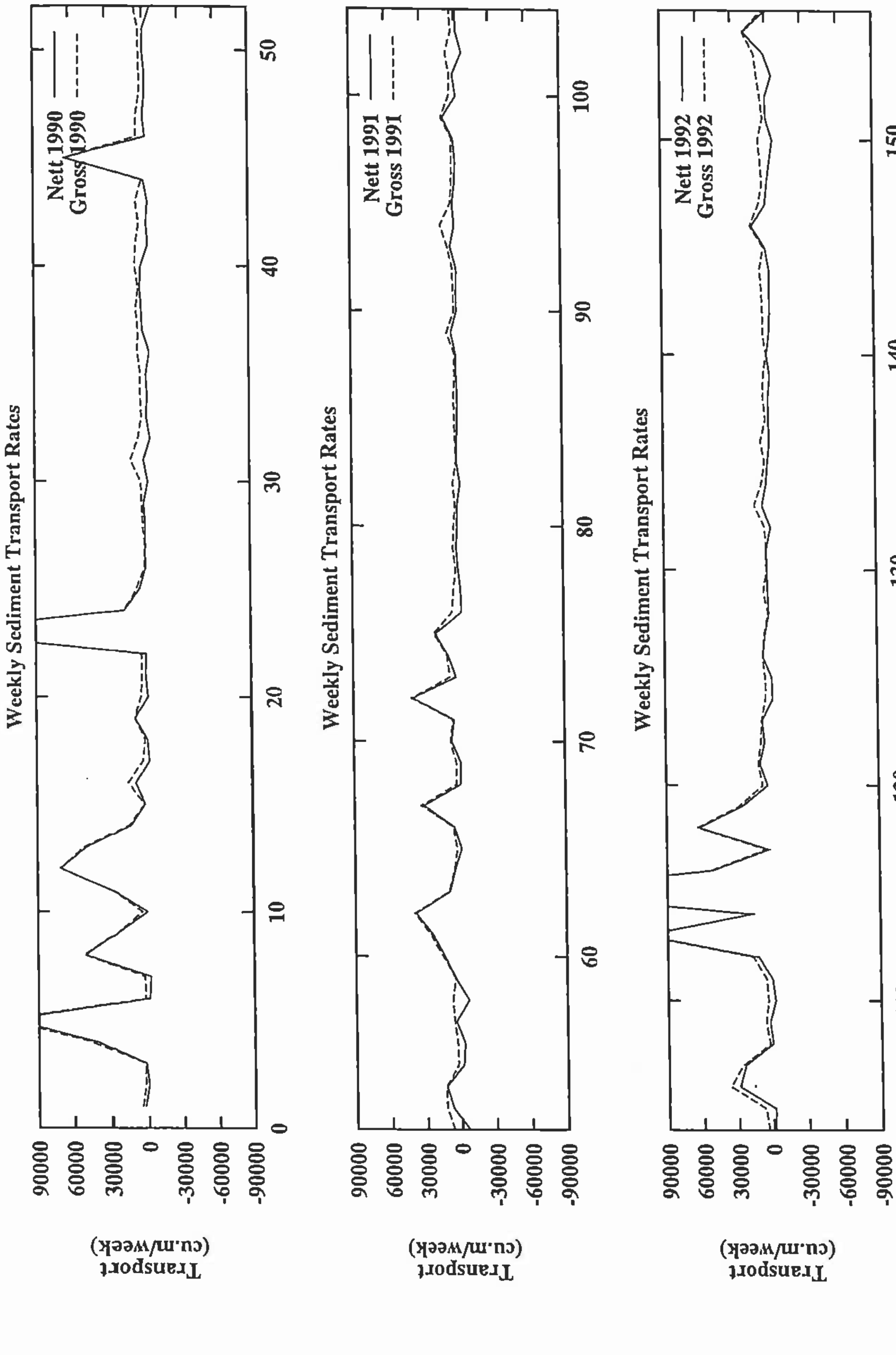
Weekly Longshore Sand Transport
Kirra



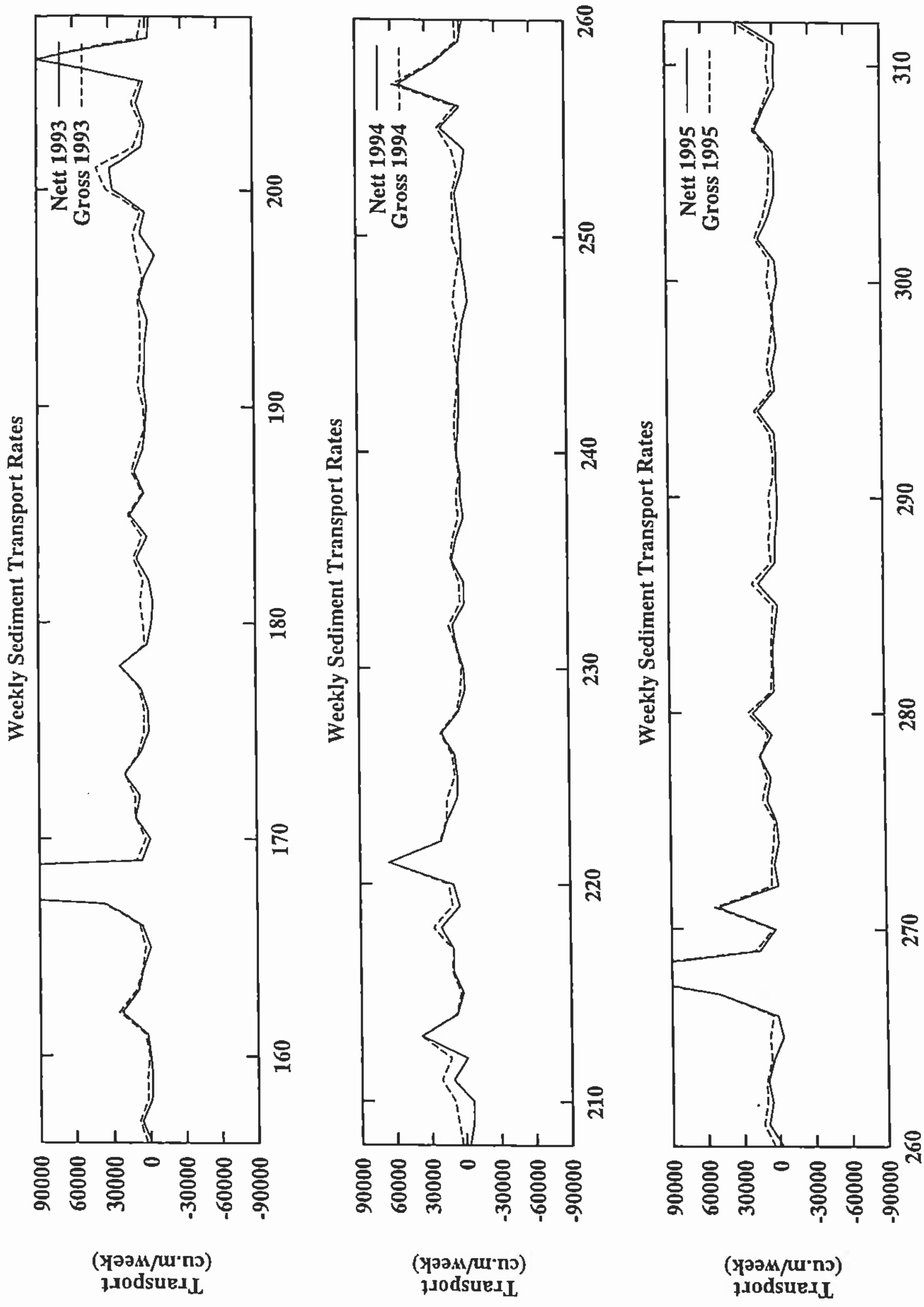
Weekly Longshore Sand Transport
Snapper Rocks



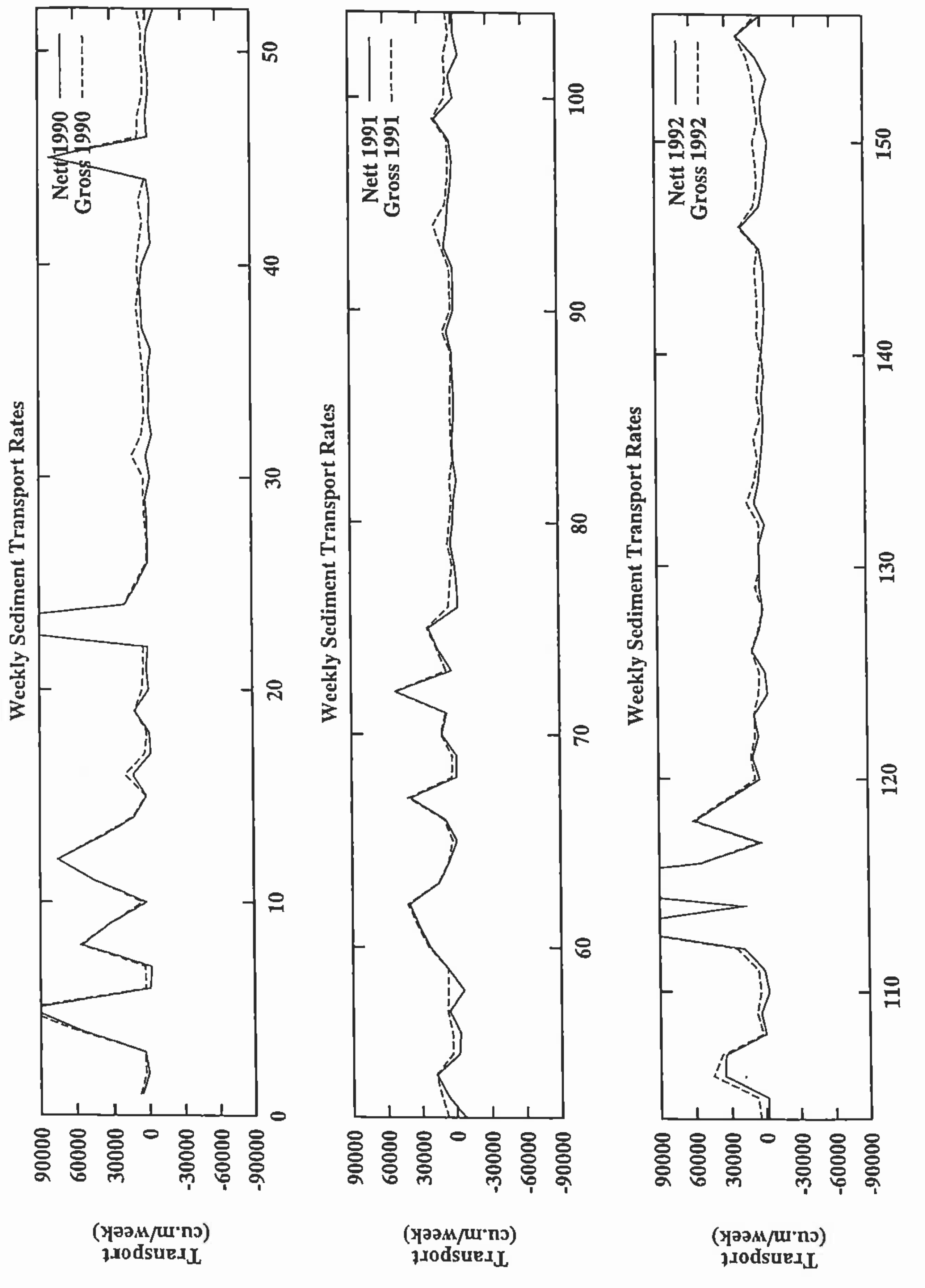
**Weekly Longshore Sand Transport
Snapper Rocks**



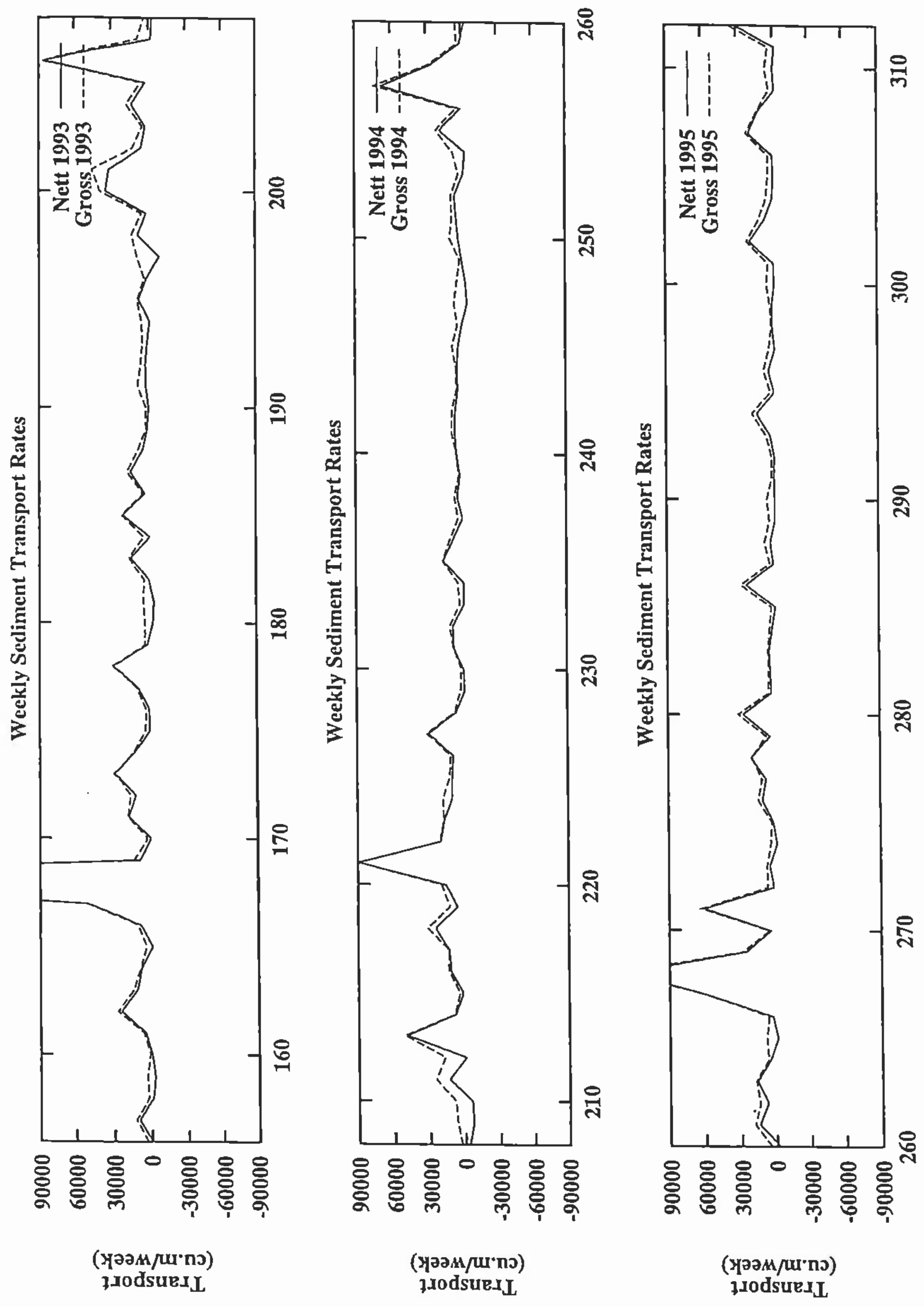
Weekly Longshore Sand Transport
Point Danger (Frog's Beach)



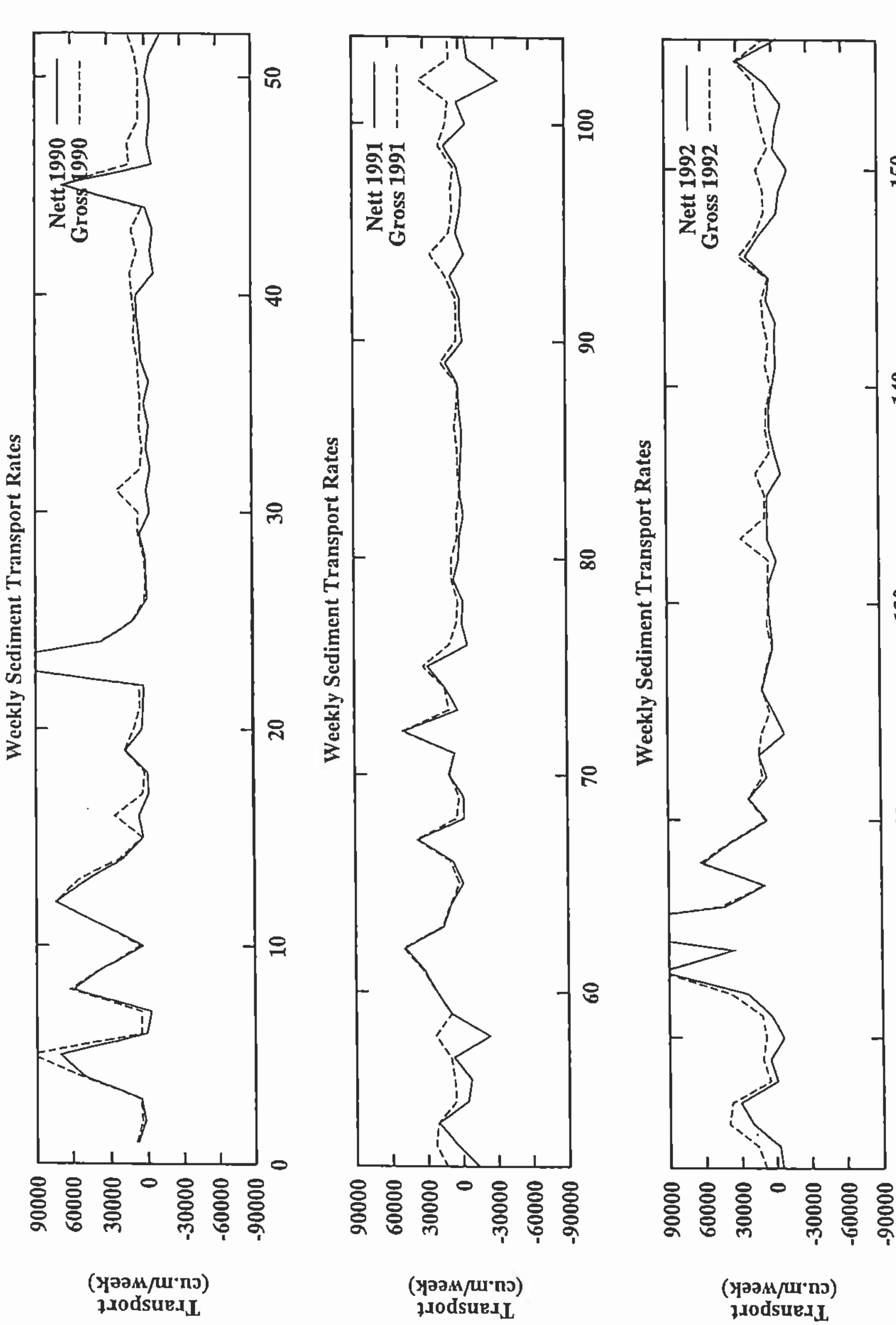
Weekly Longshore Sand Transport
Point Danger (Frog's Beach)



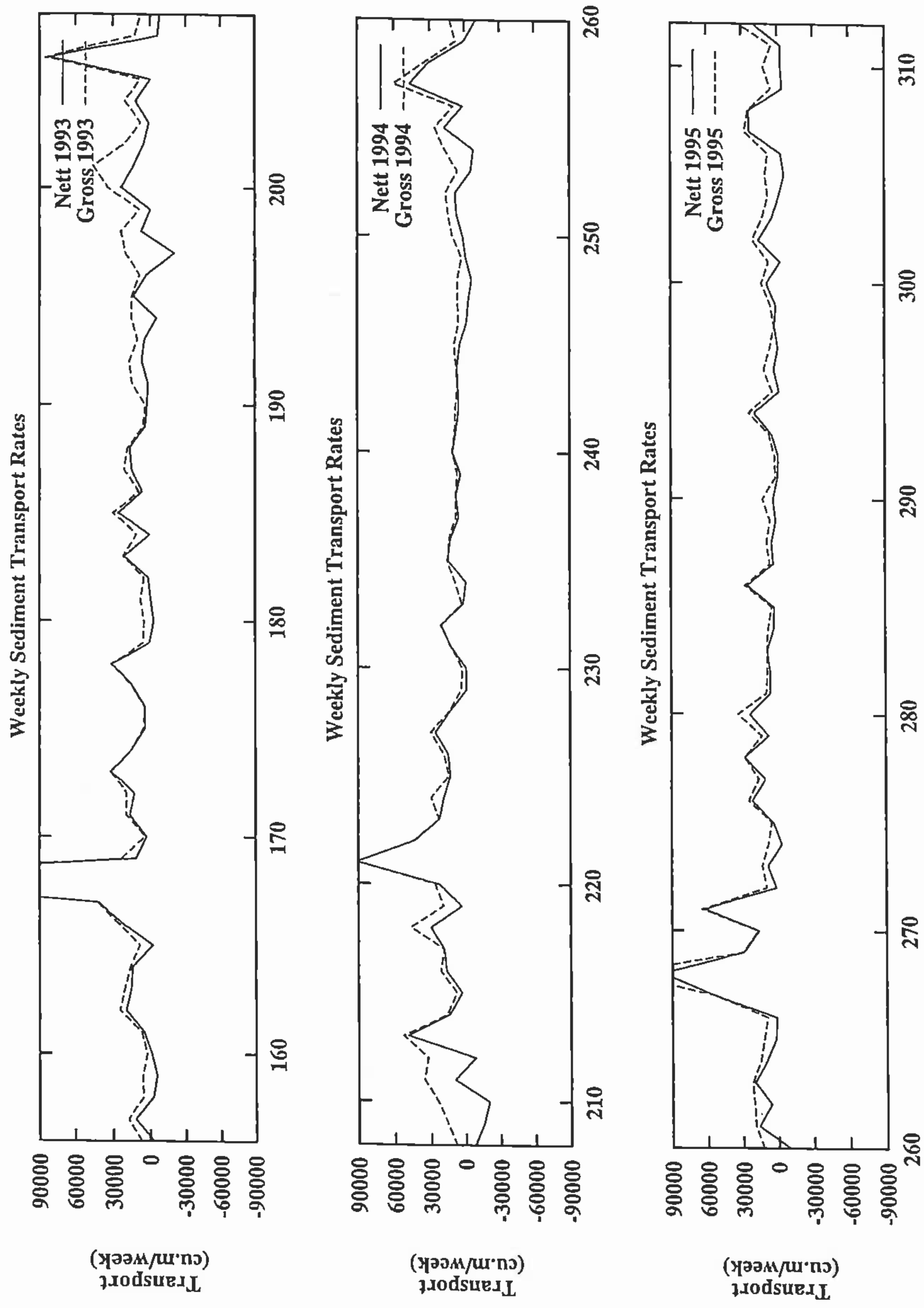
Weekly Longshore Sand Transport Lovers Rock



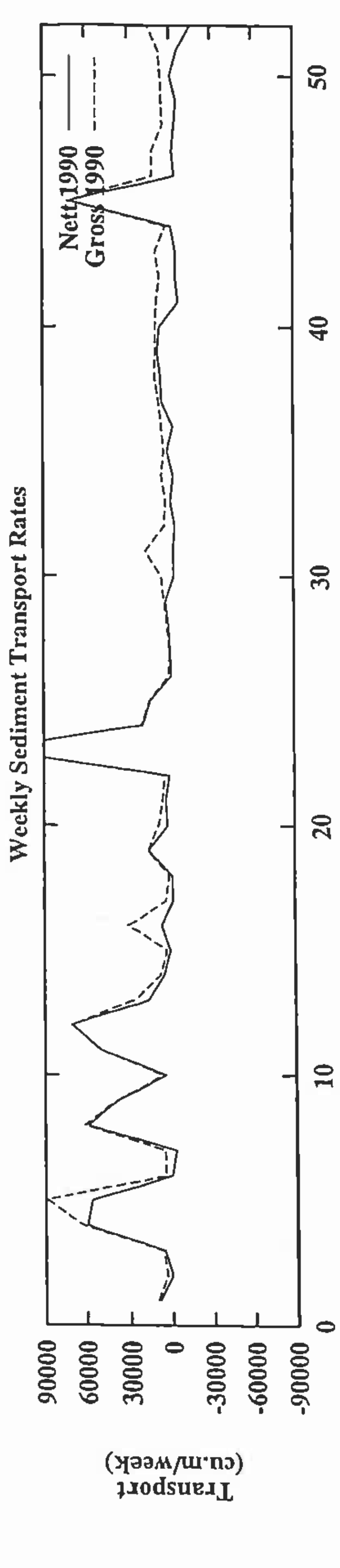
Weekly Longshore Sand Transport
Lovers Rock

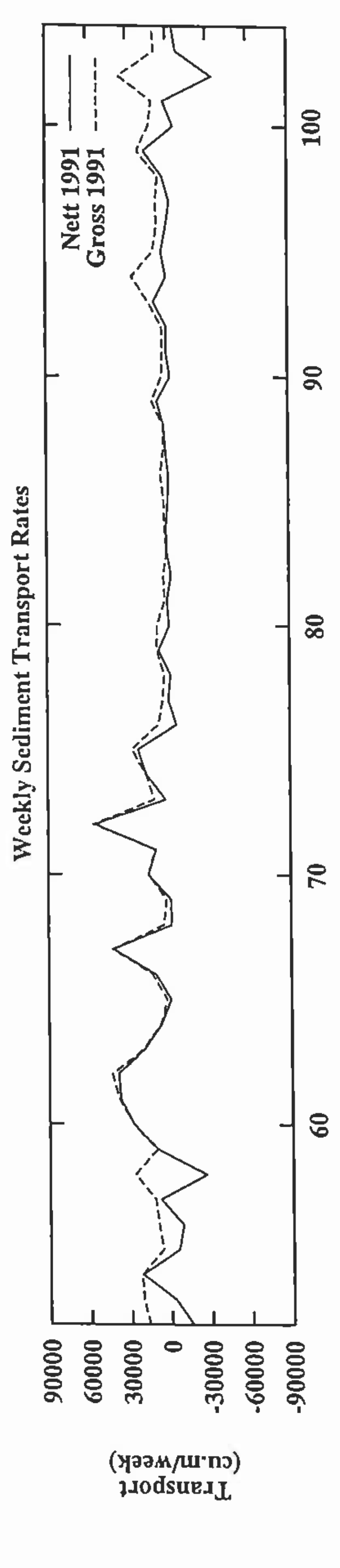


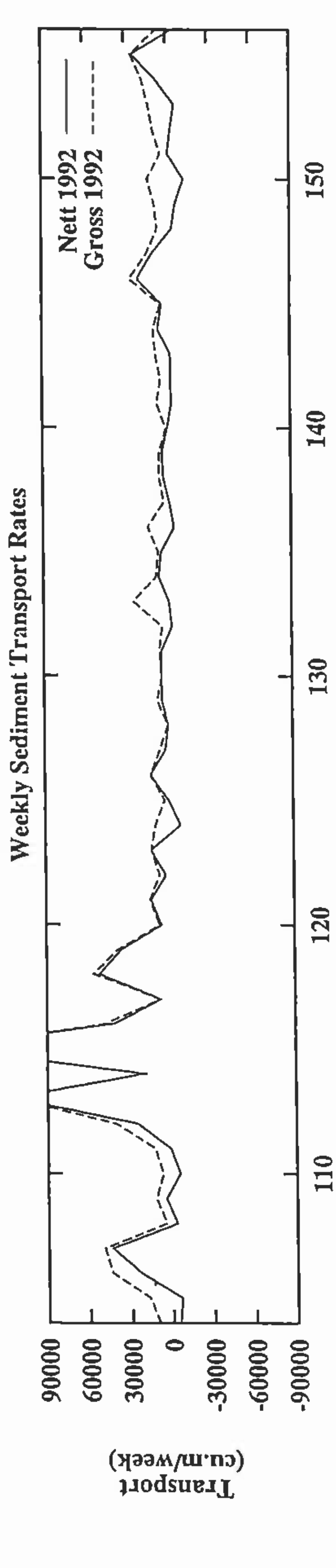
Weekly Longshore Sand Transport
Duranbah



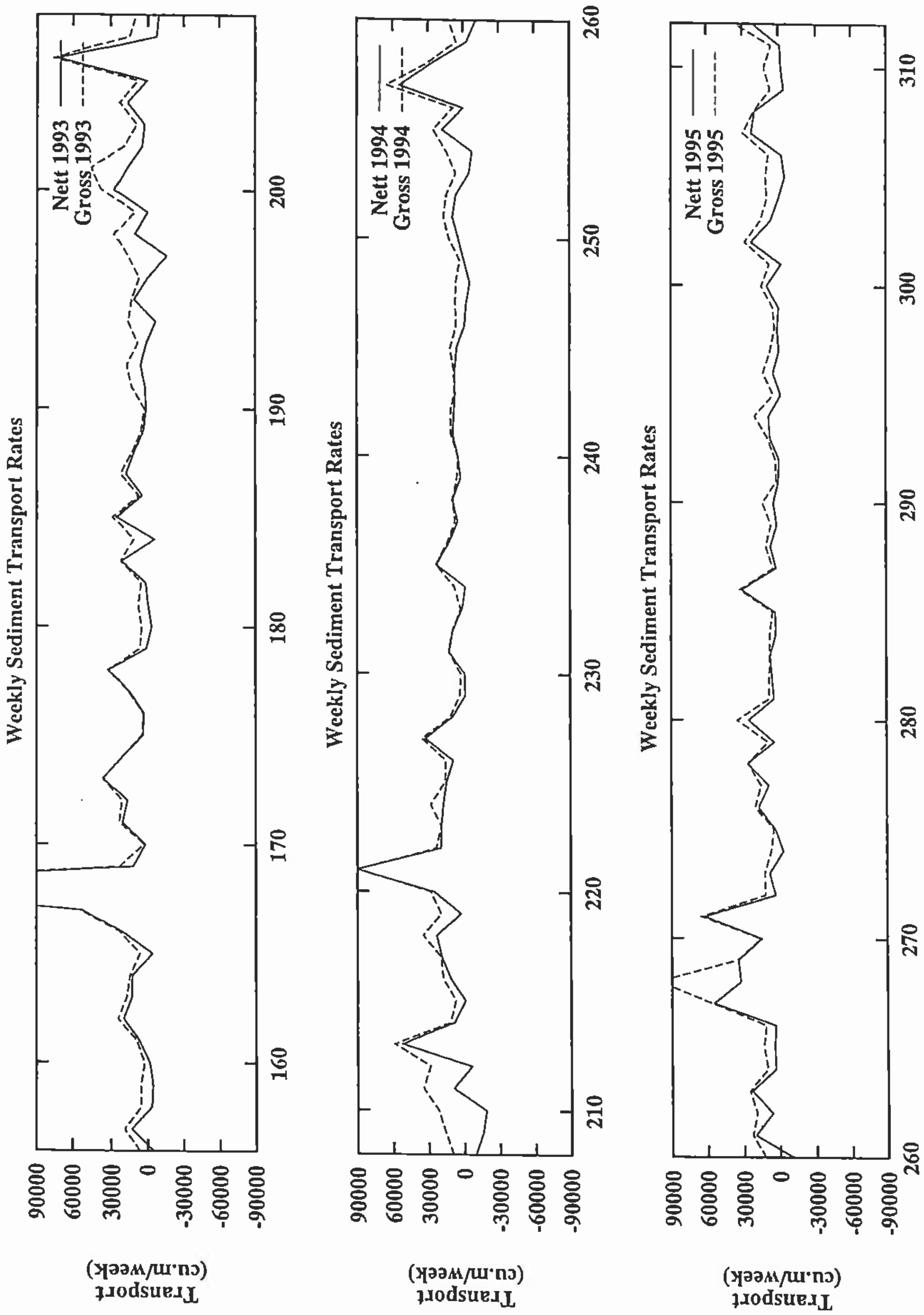
Weekly Longshore Sand Transport
Duranbah



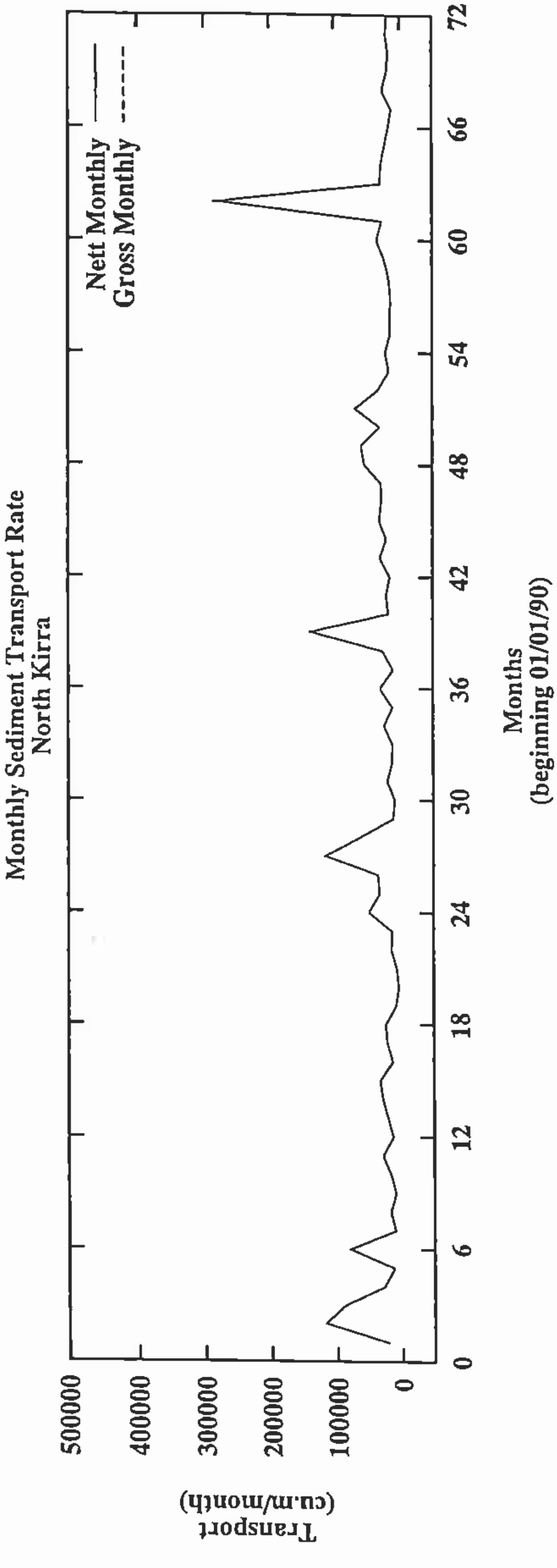


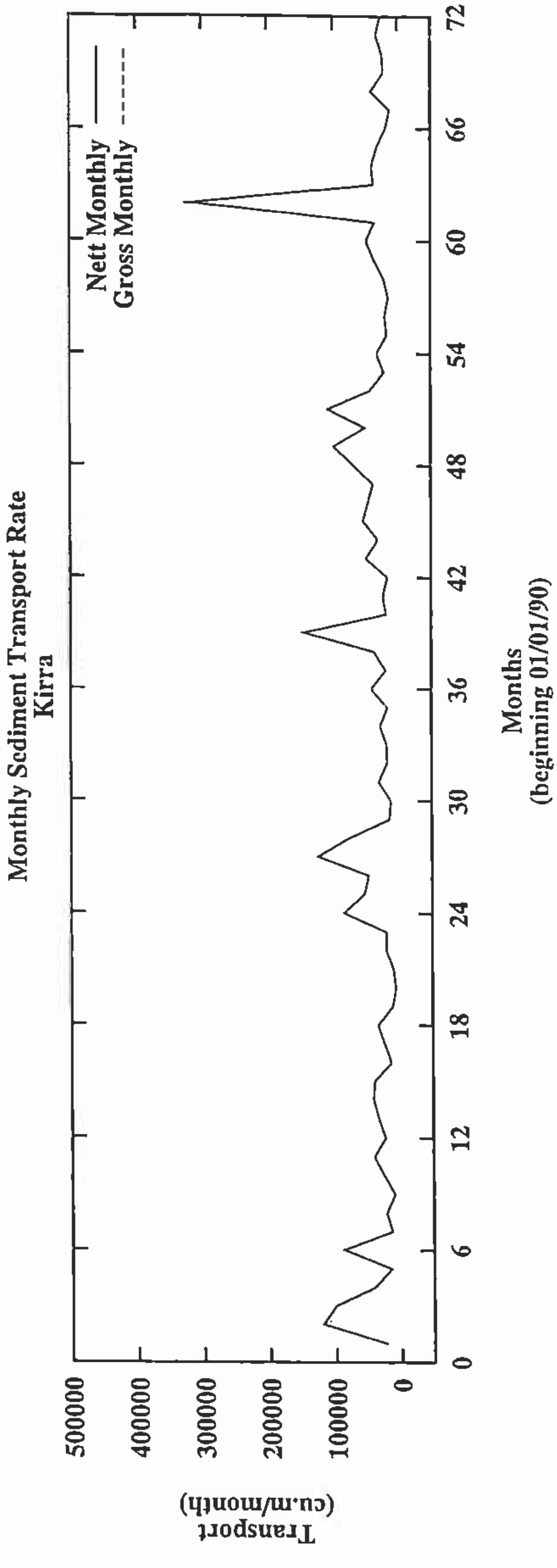


Weekly Longshore Sand Transport
Letitia Spit

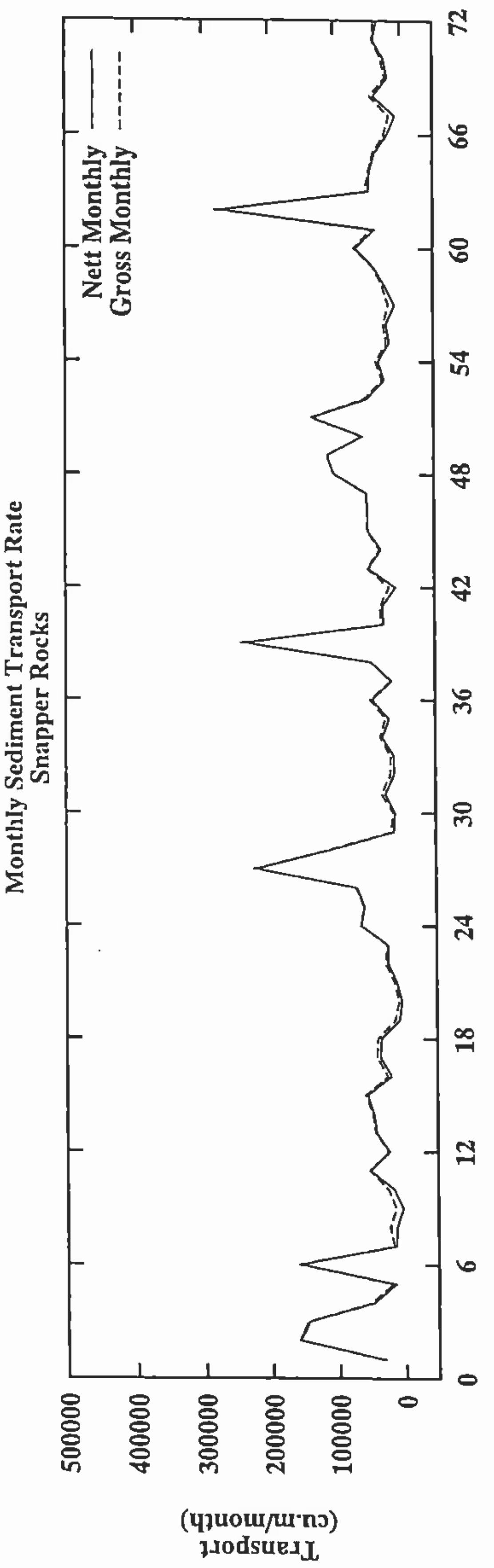


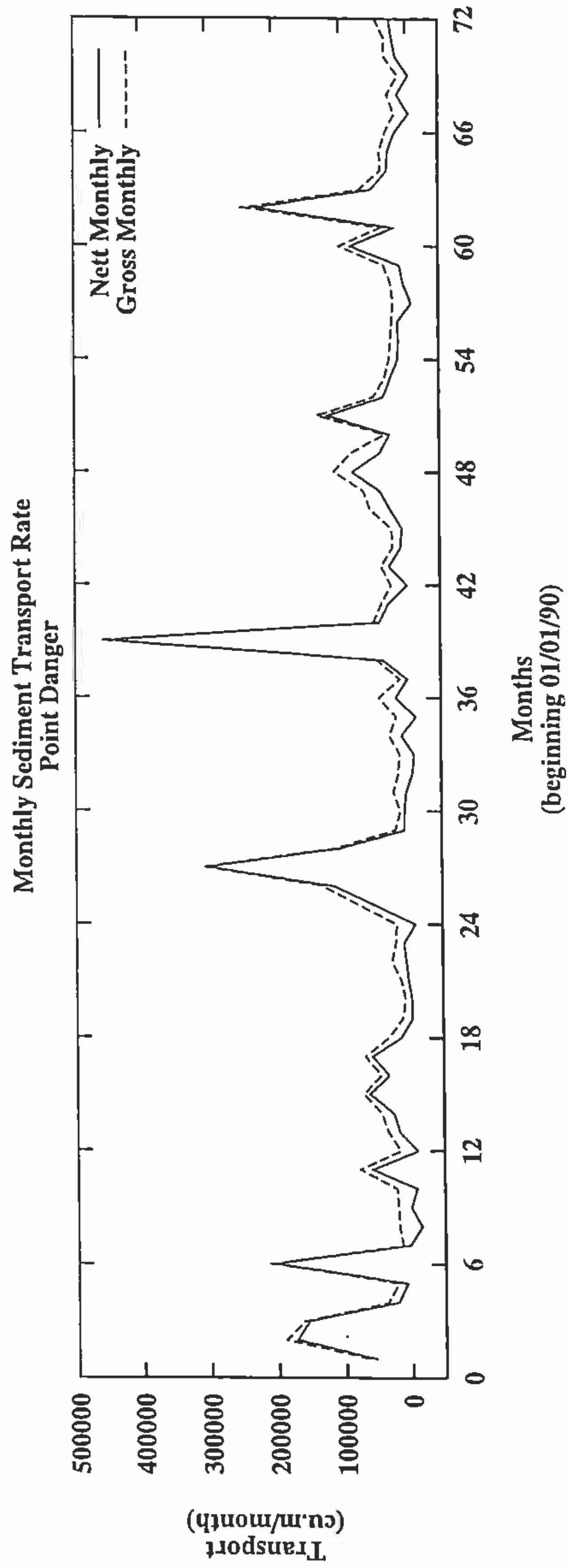
Weekly Longshore Sand Transport Letitia Spit



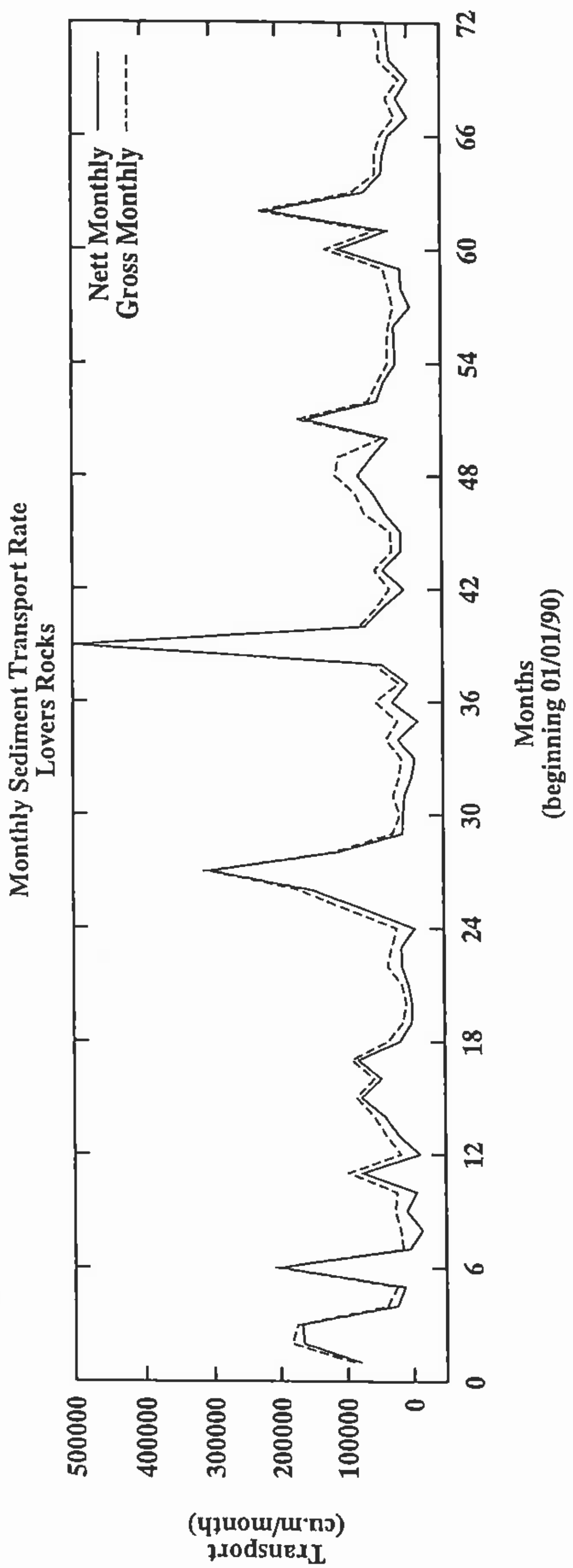


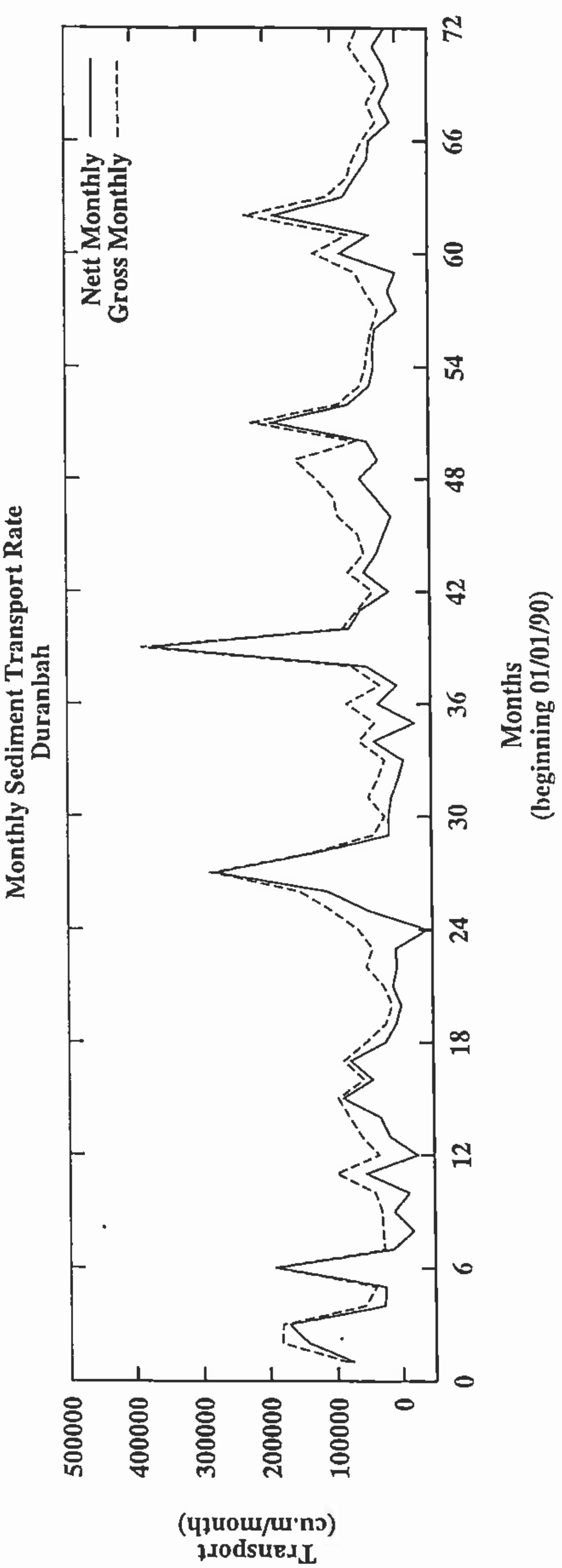
Monthly Longshore Sand Transport North Kirra and Kirra



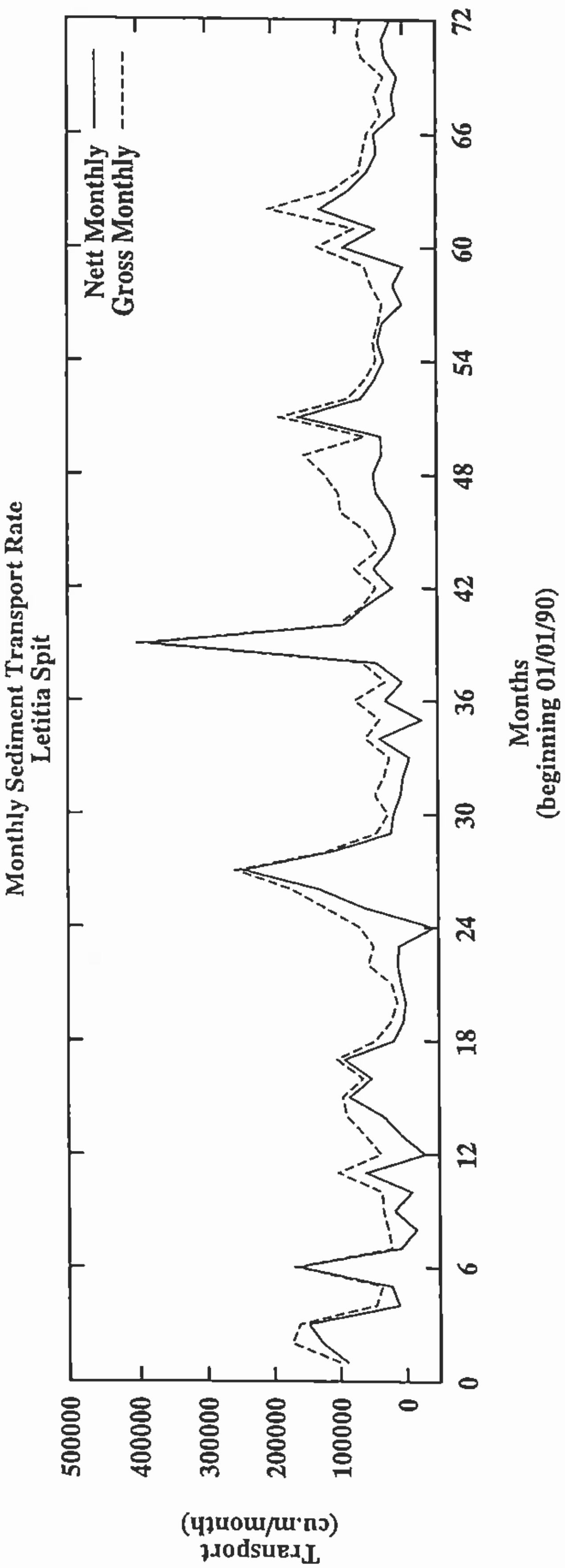


Monthly Longshore Sand Transport
Snapper Rocks & Point Danger (Frog's Beach)



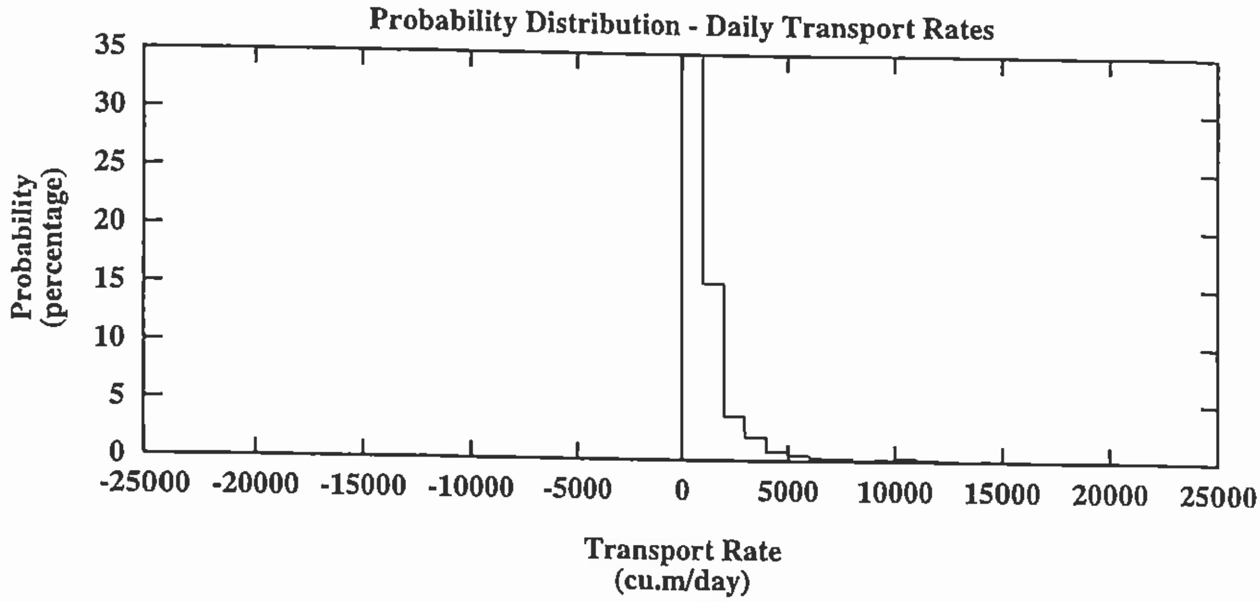


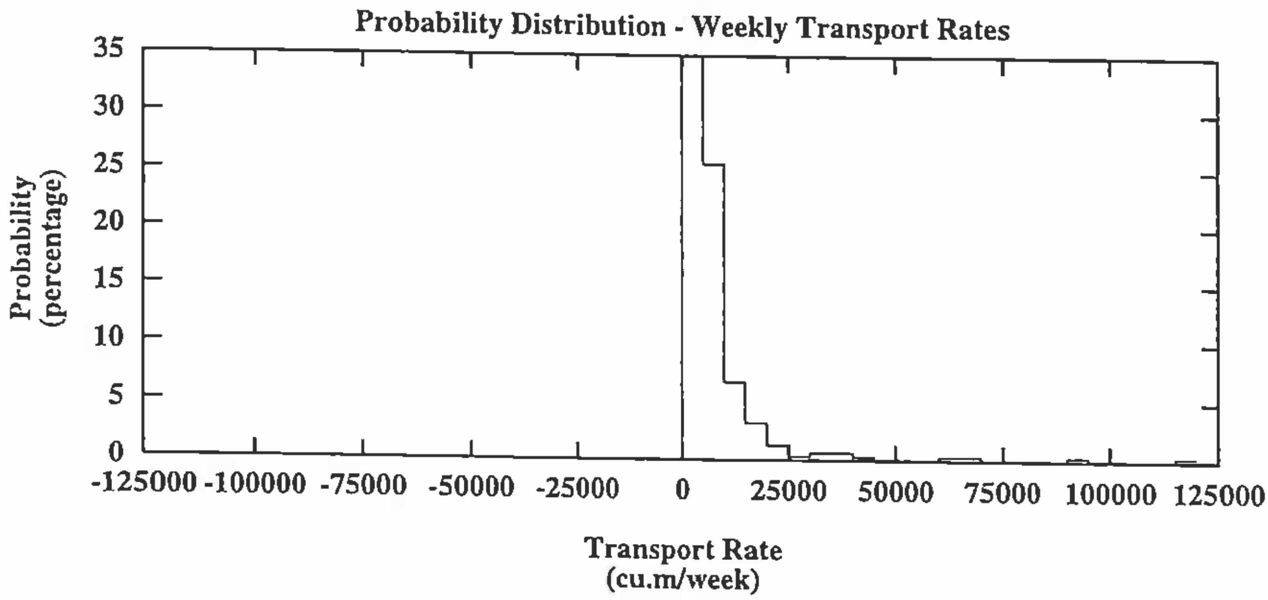
Monthly Longshore Sand Transport
Lovers Rock & Duranbah

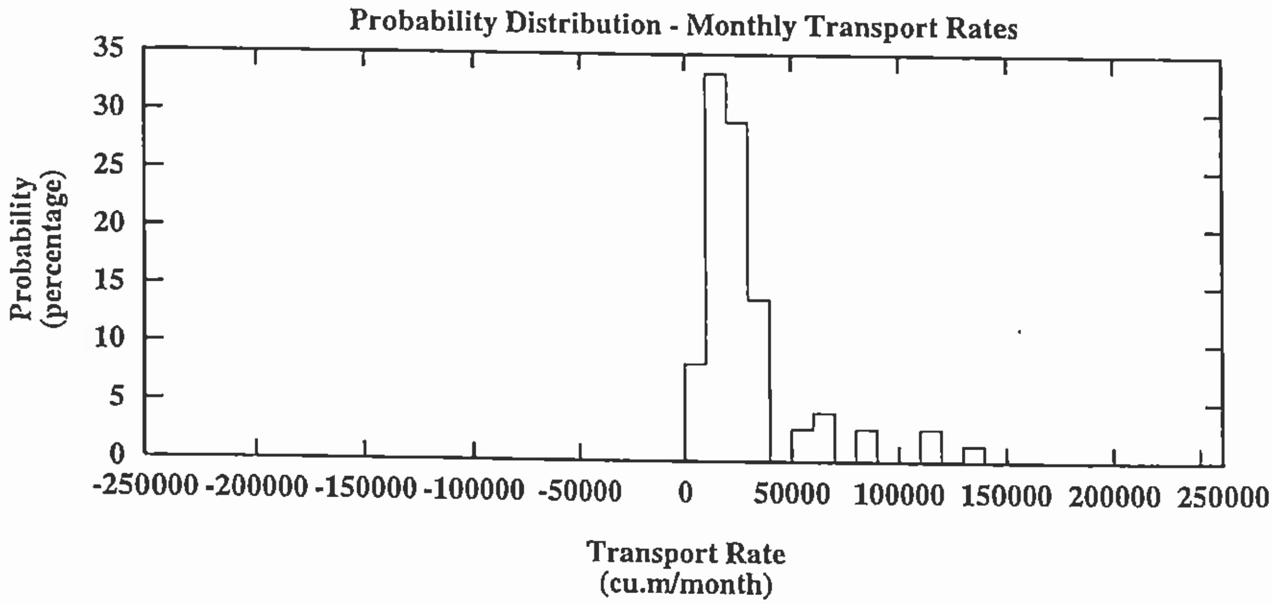


Monthly Longshore Sand Transport Letitia Spit

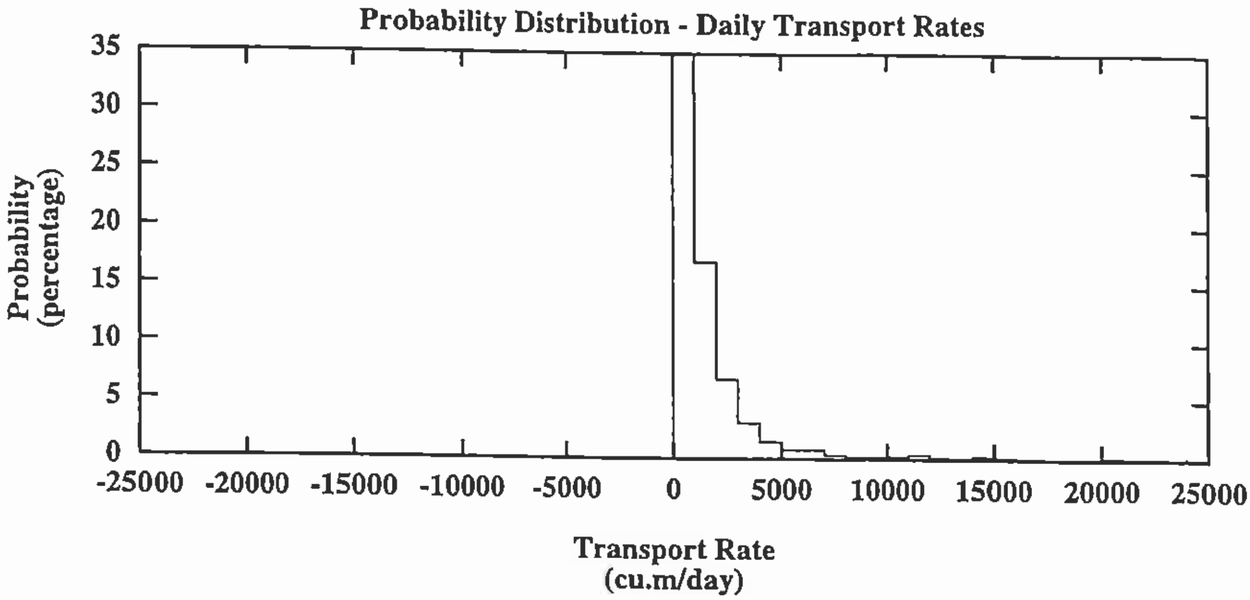
APPENDIX C: Longshore Transport Occurrence Probabilities

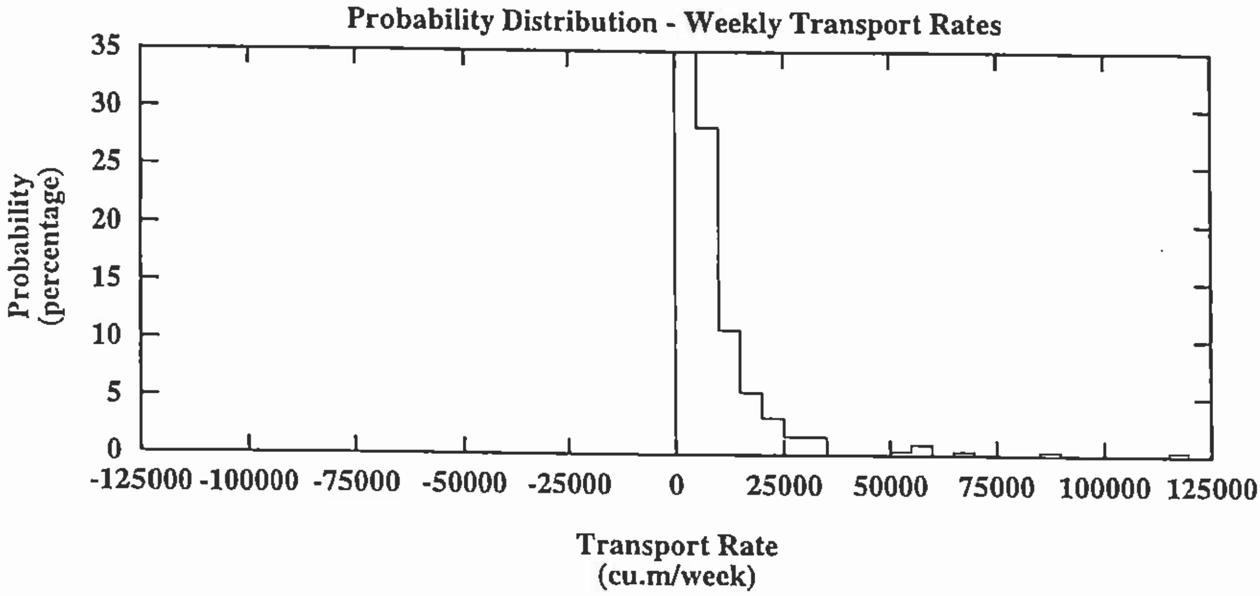


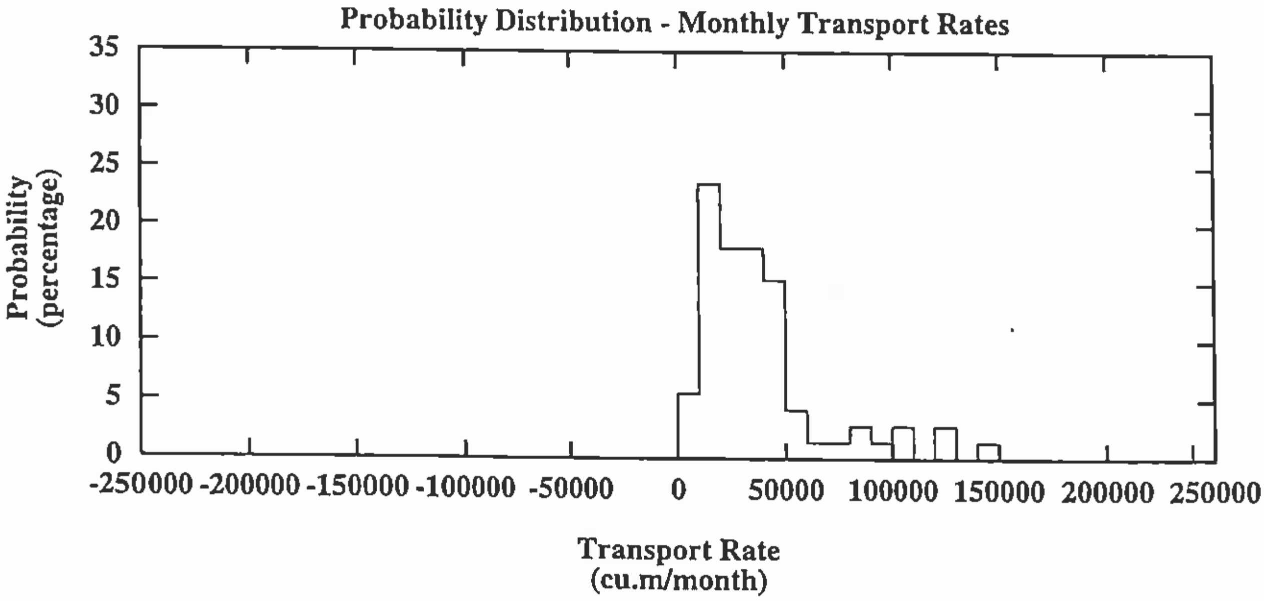


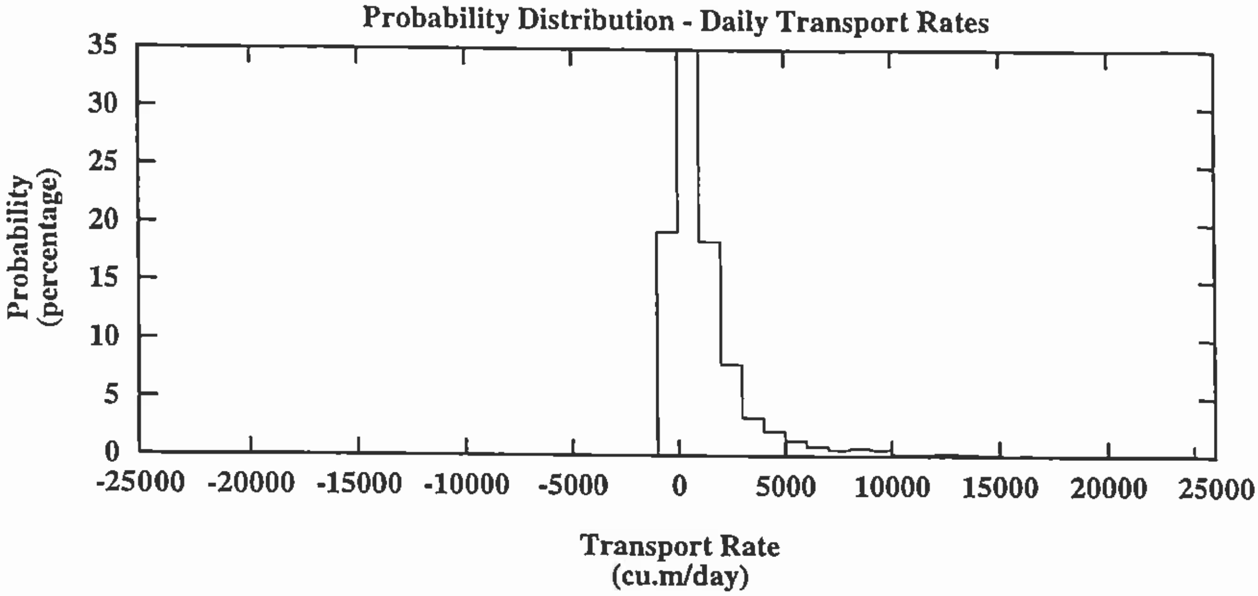


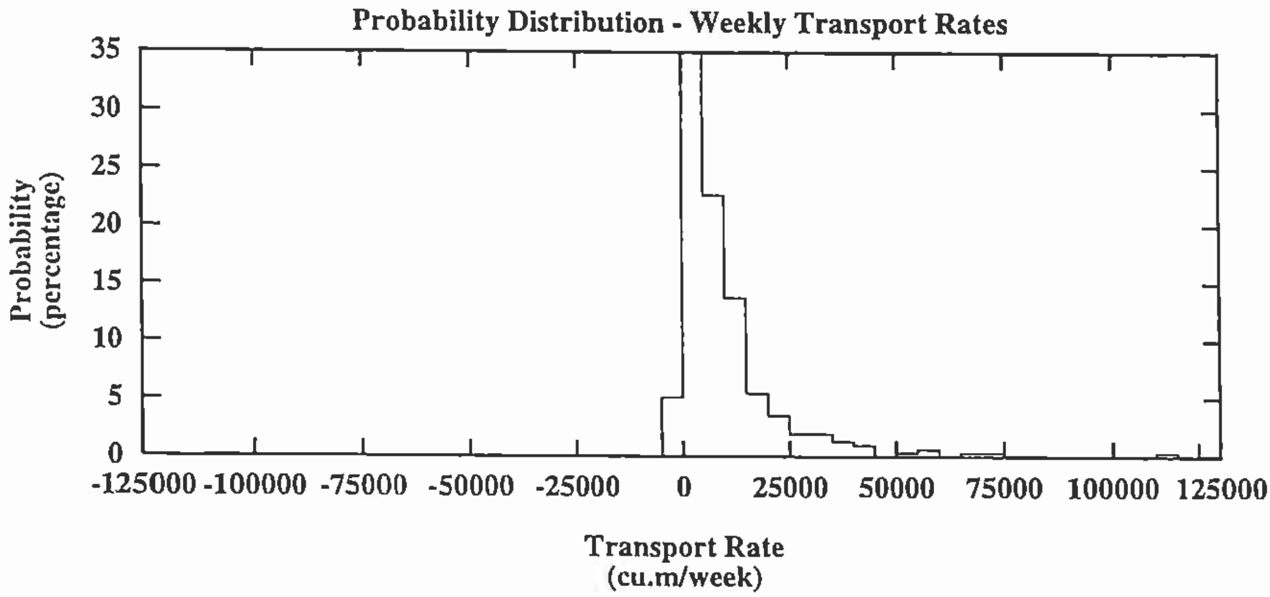
Longshore Sand Transport Probabilities
North Kirra

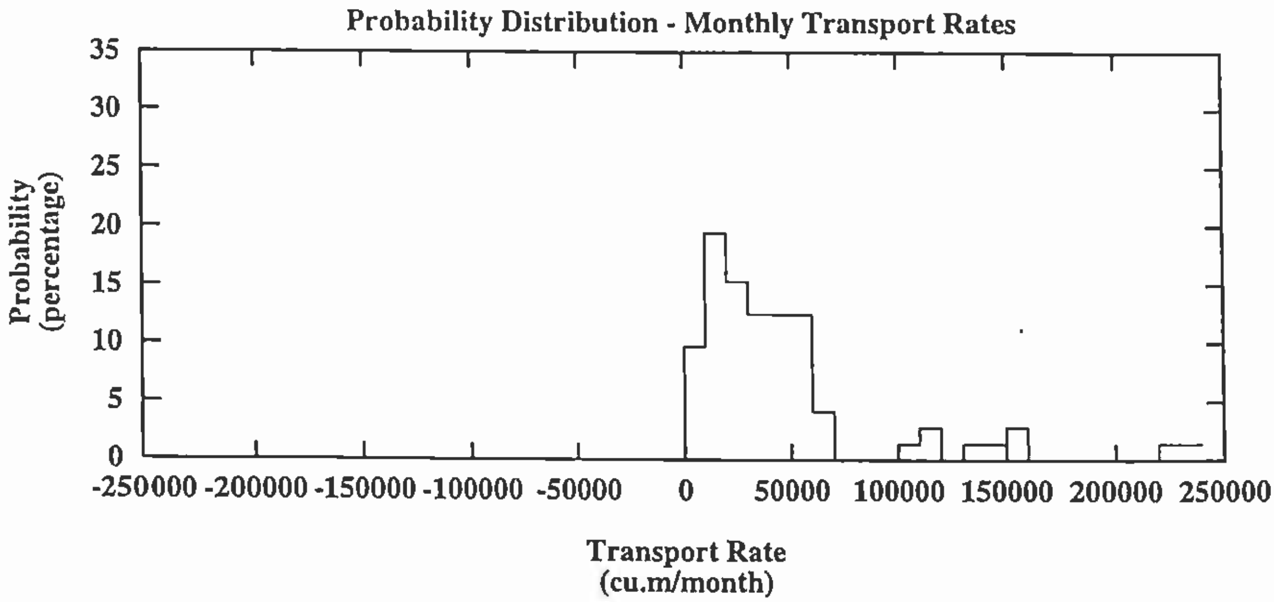

Probability Distribution - Daily Transport Rates
Probability (percentage)
35
30
25
20
15
10
5
0
-25000
-20000
-15000
-10000
-5000
0
5000
10000
15000
20000
25000
Transport Rate (cu.m/day)


Probability Distribution - Weekly Transport Rates
Probability (percentage)
35
30
25
20
15
10
5
0
-125000
-100000
-75000
-50000
-25000
0
25000
50000
75000
100000
125000
Transport Rate (cu.m/week)


Probability Distribution - Monthly Transport Rates
Probability (percentage)
35
30
25
20
15
10
5
0
-250000
-200000
-150000
-100000
-50000
0
50000
100000
150000
200000
250000
Transport Rate (cu.m/month)

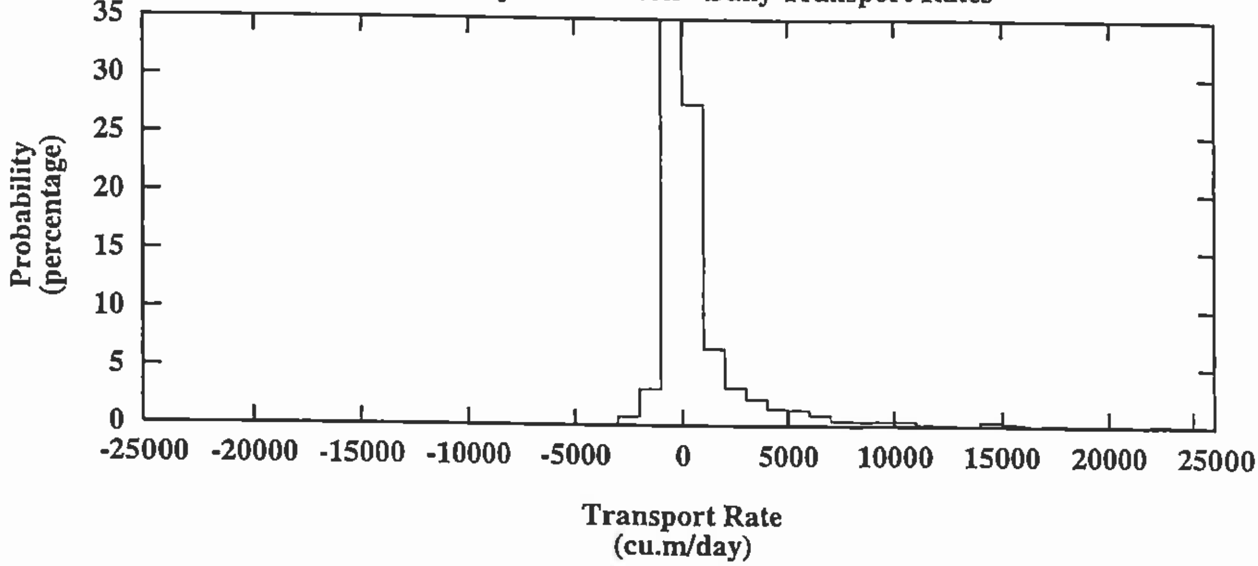




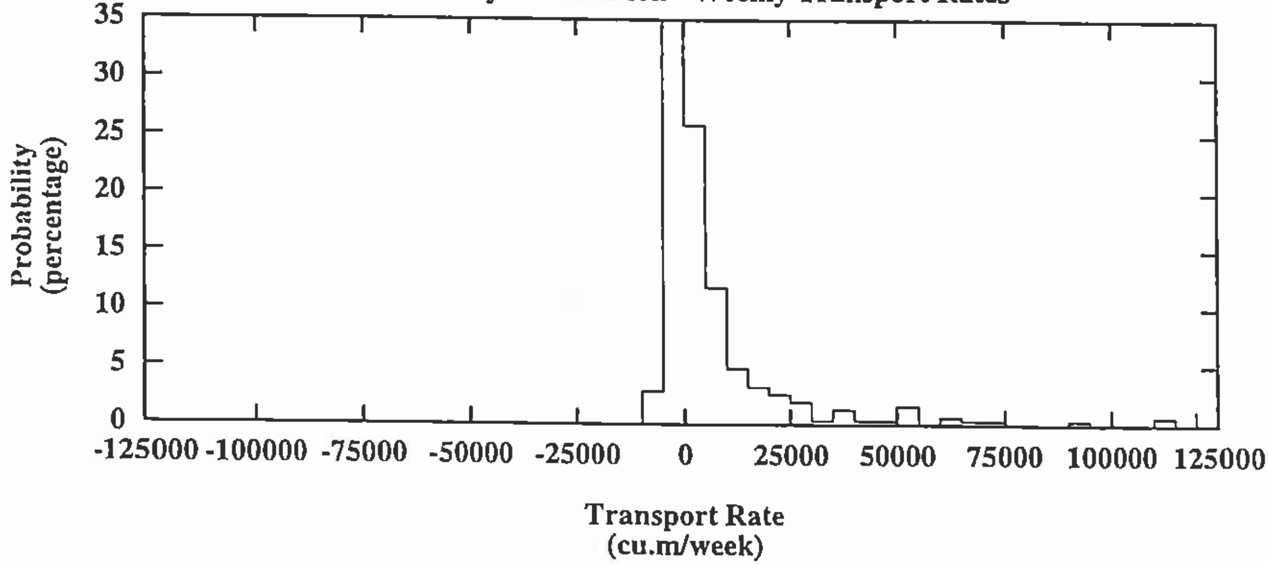


Longshore Sand Transport Probabilities Snapper Rocks

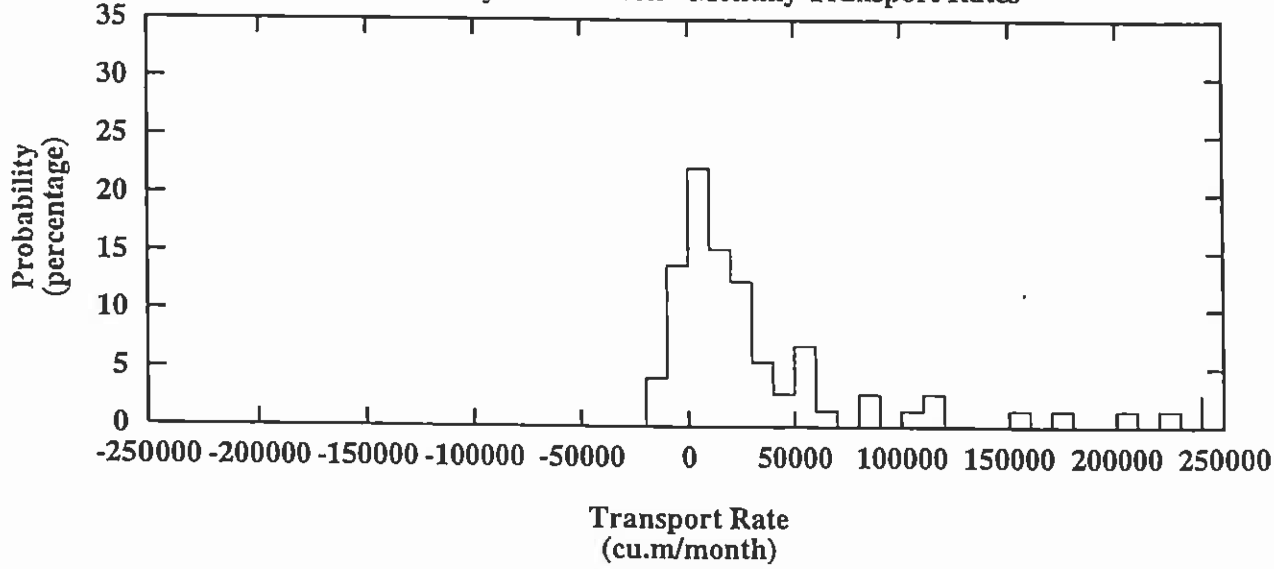
Probability Distribution - Daily Transport Rates



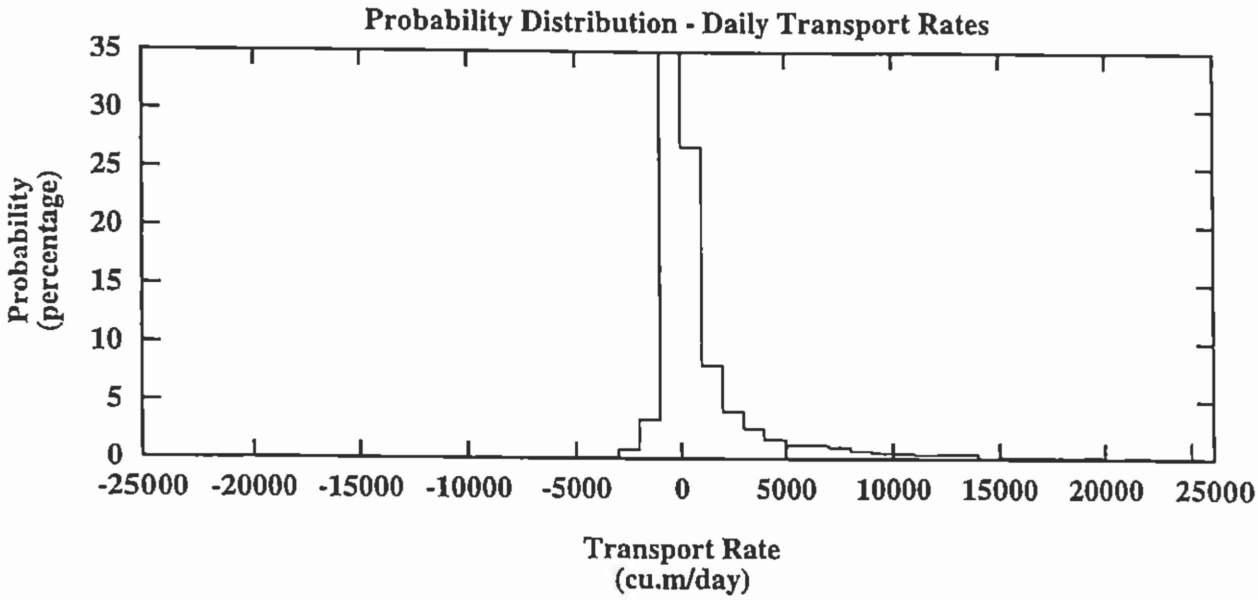
Probability Distribution - Weekly Transport Rates

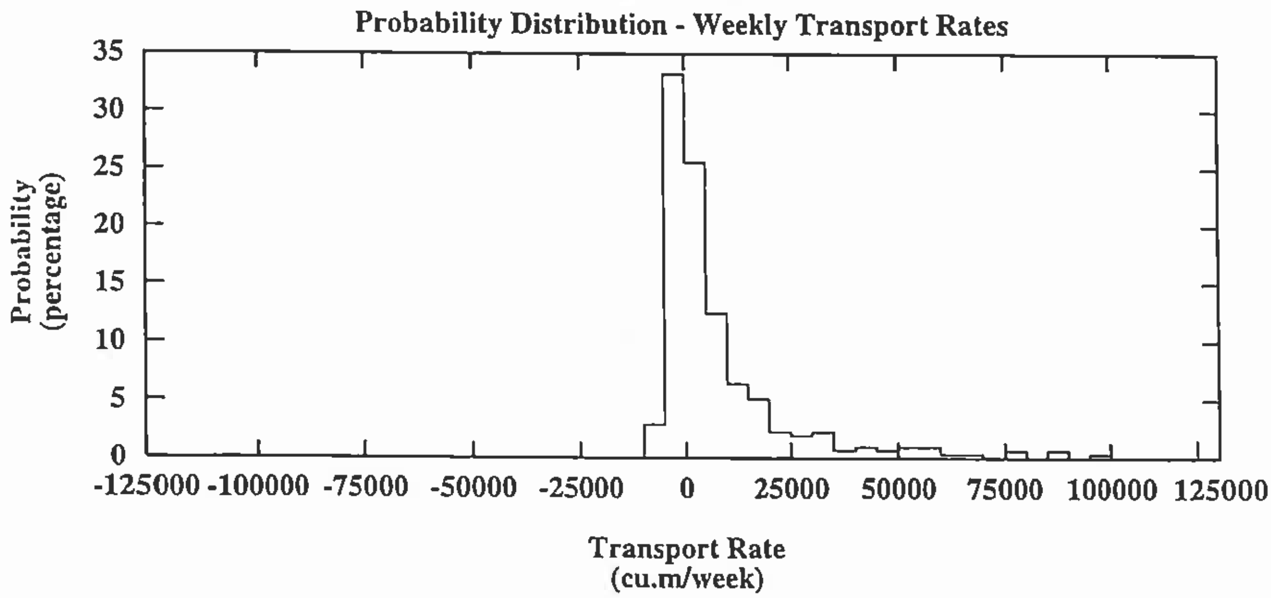


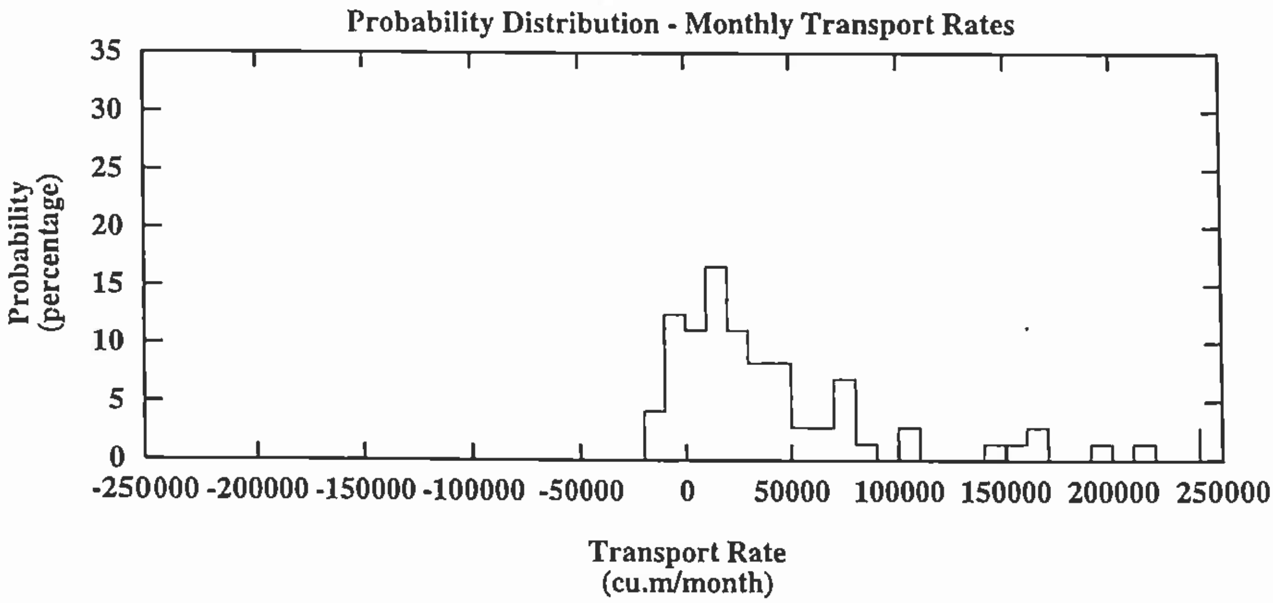
Probability Distribution - Monthly Transport Rates



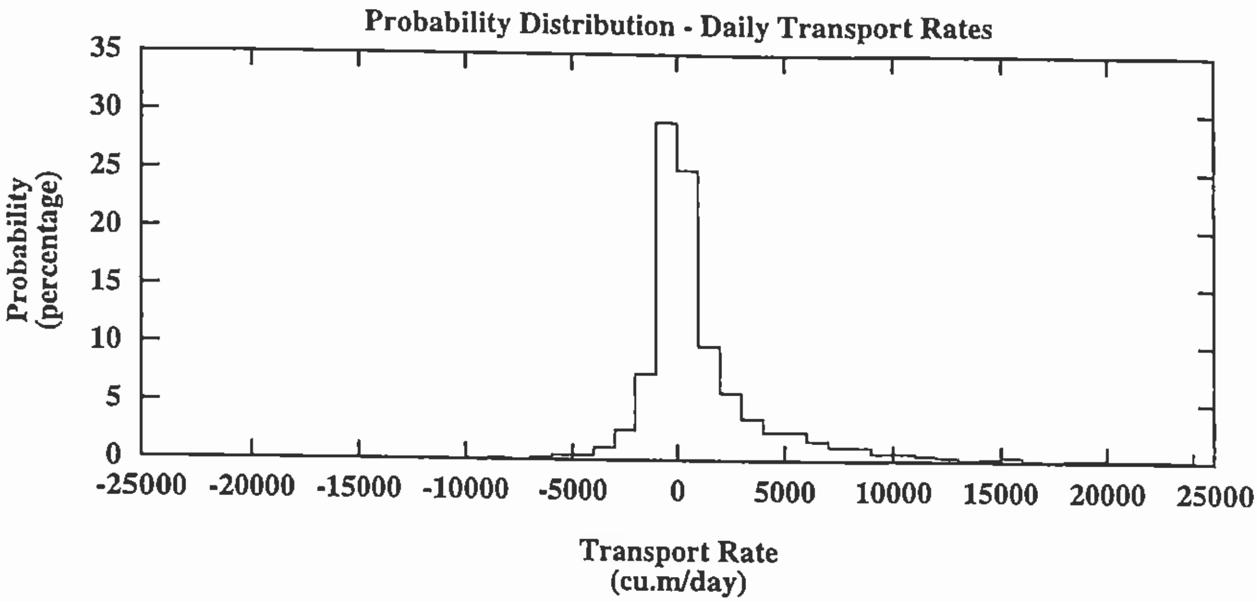
Longshore Sand Transport Probabilities
Point Danger (Frog's Beach)

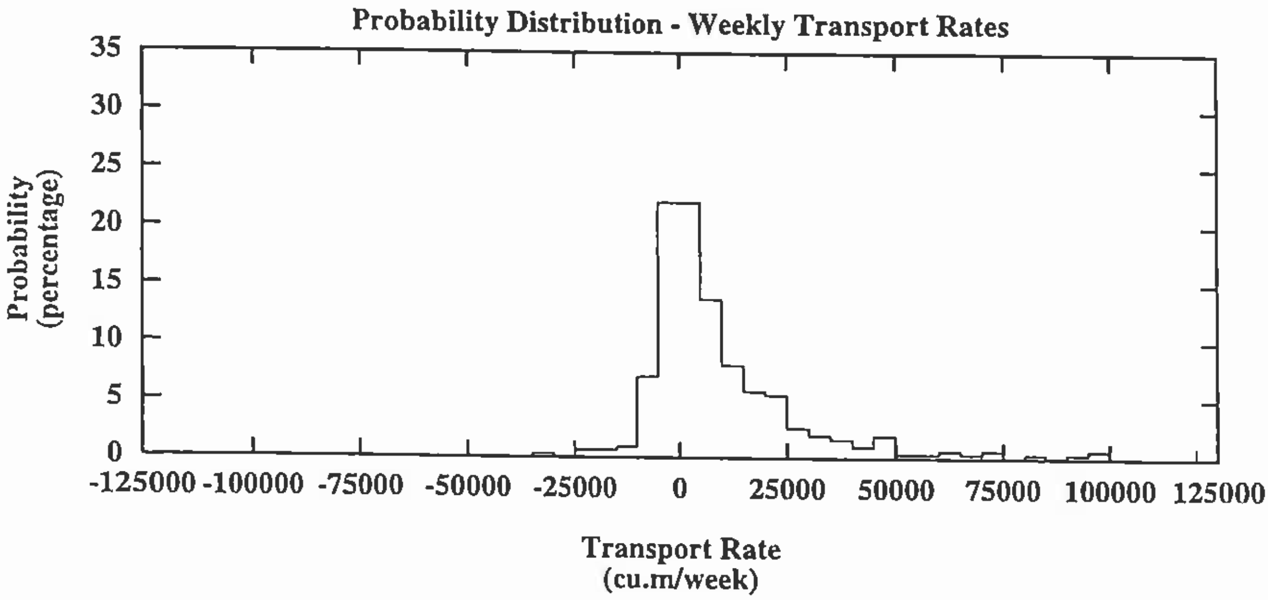


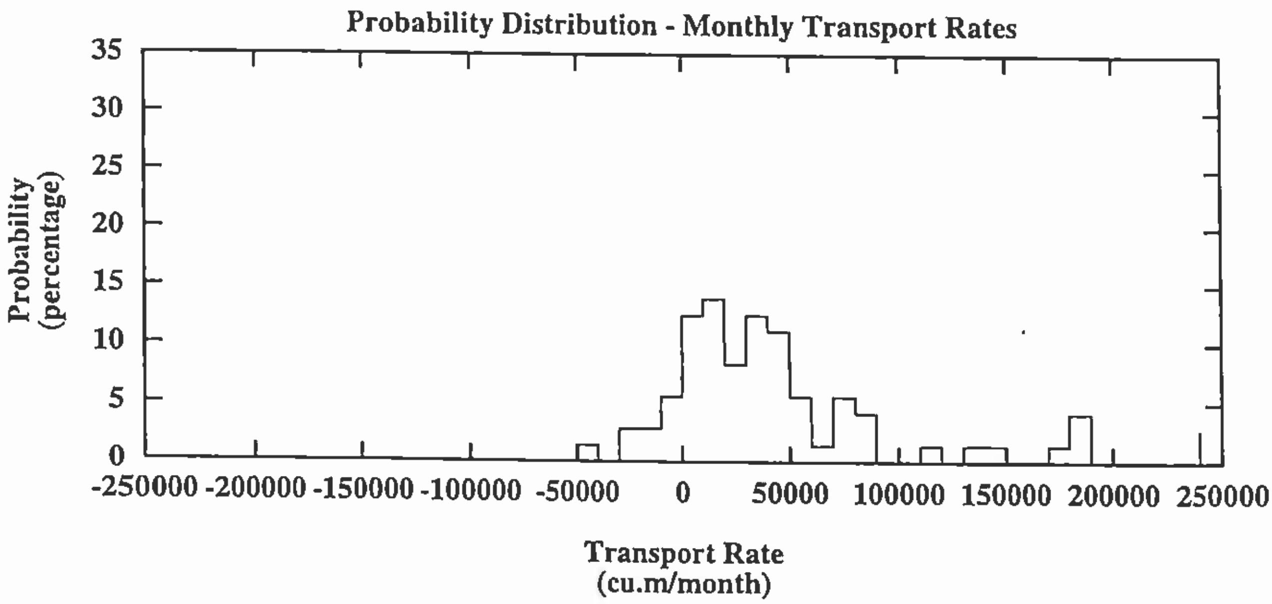




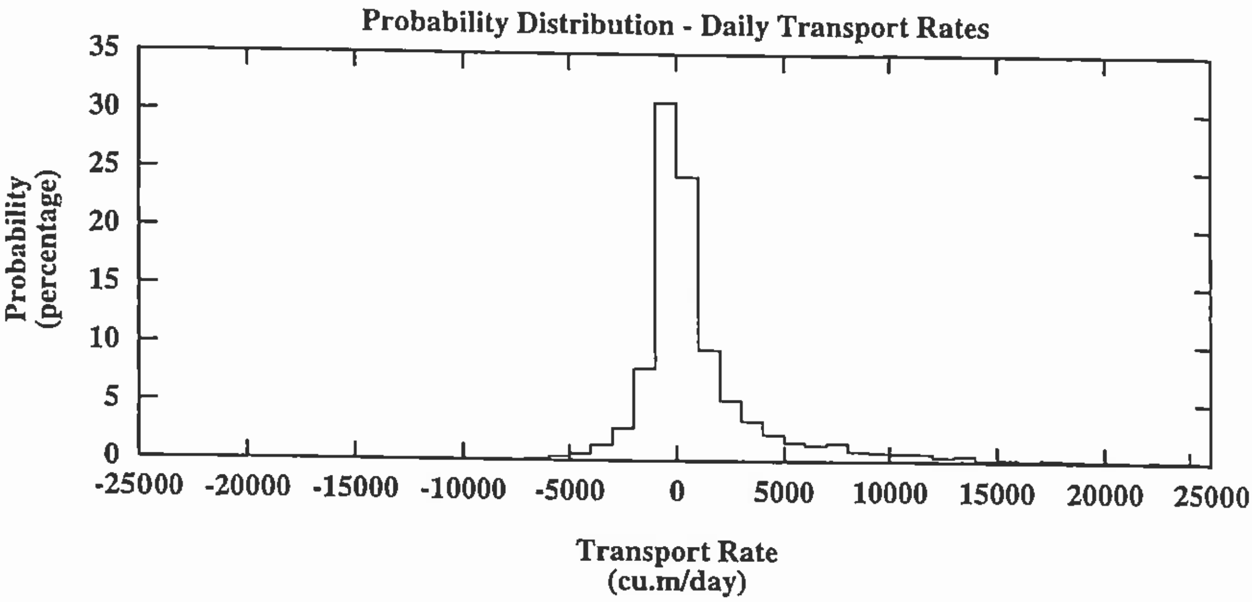
Longshore Sand Transport Probabilities
Lovers Rock

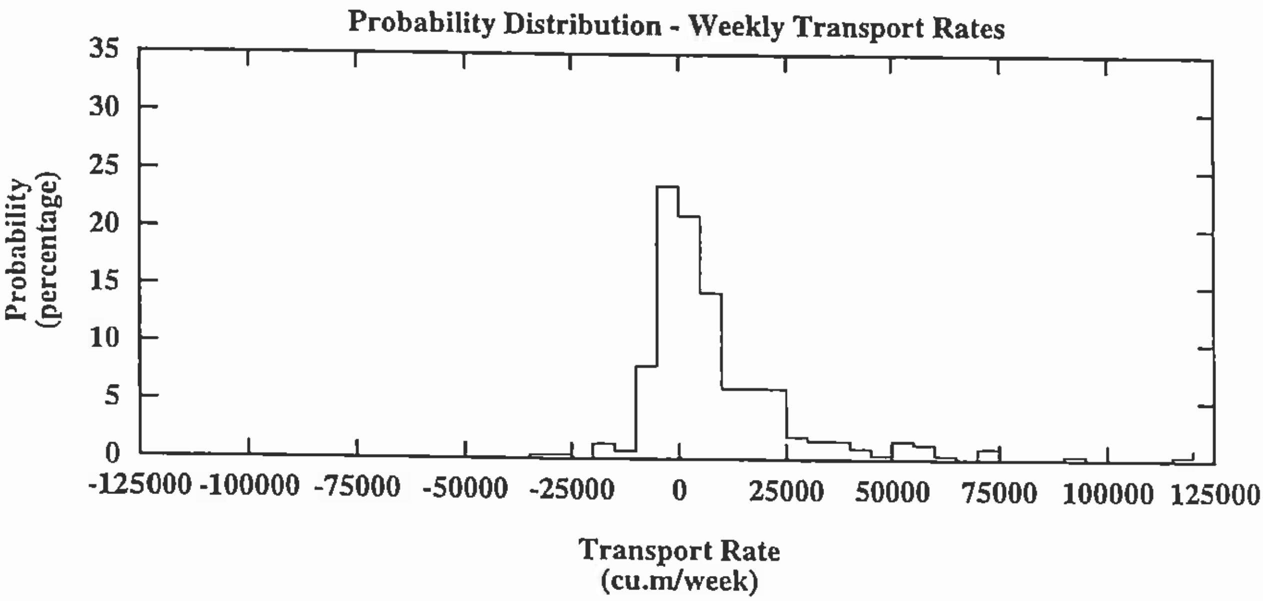


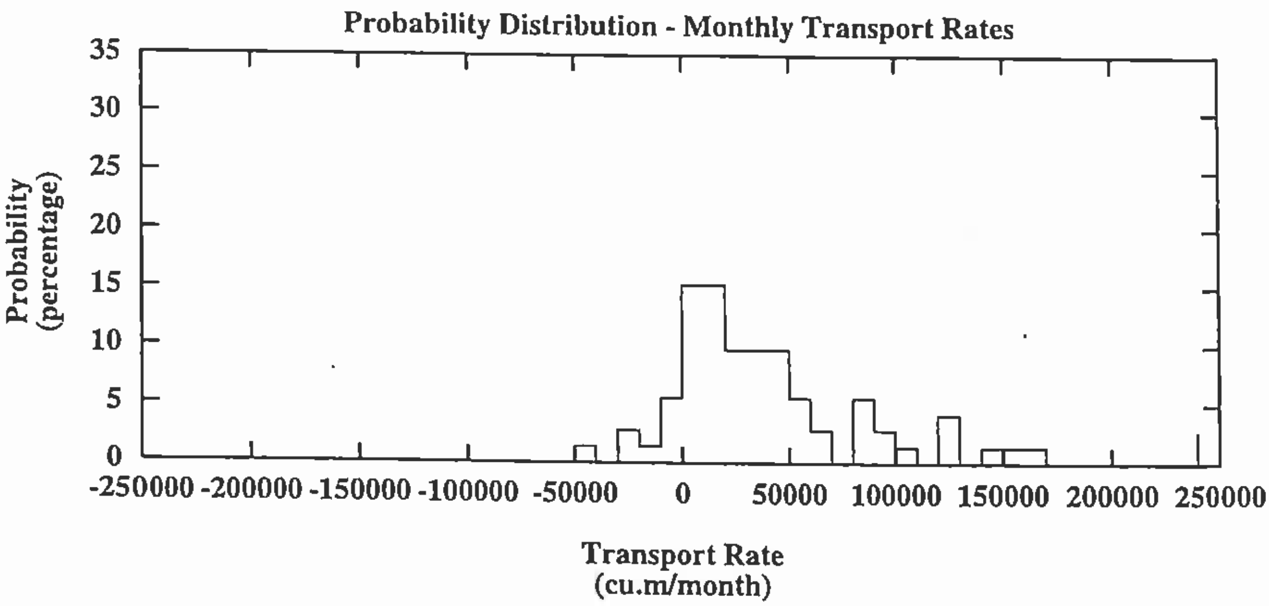




Longshore Sand Transport Probabilities
Duranbah

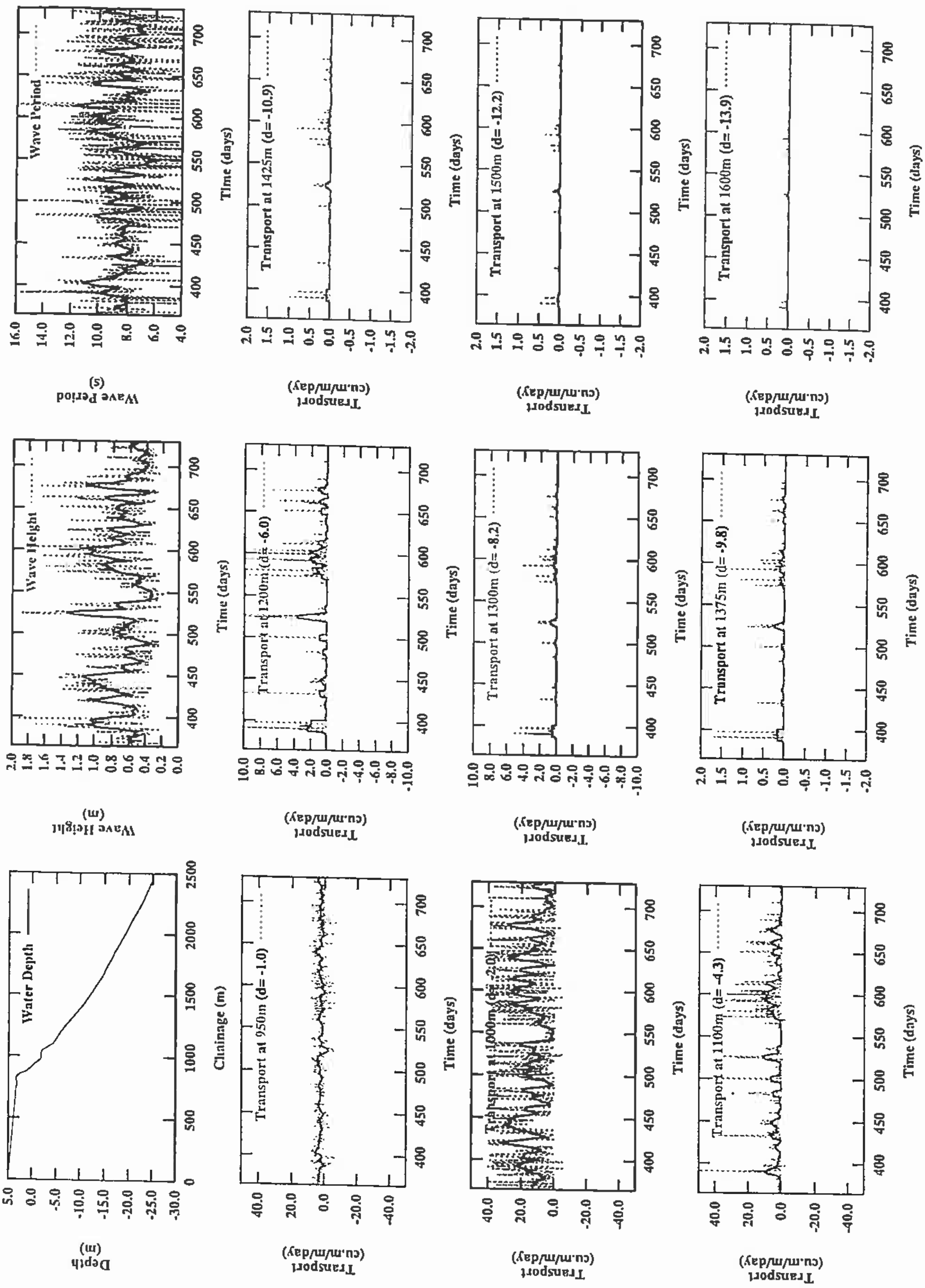






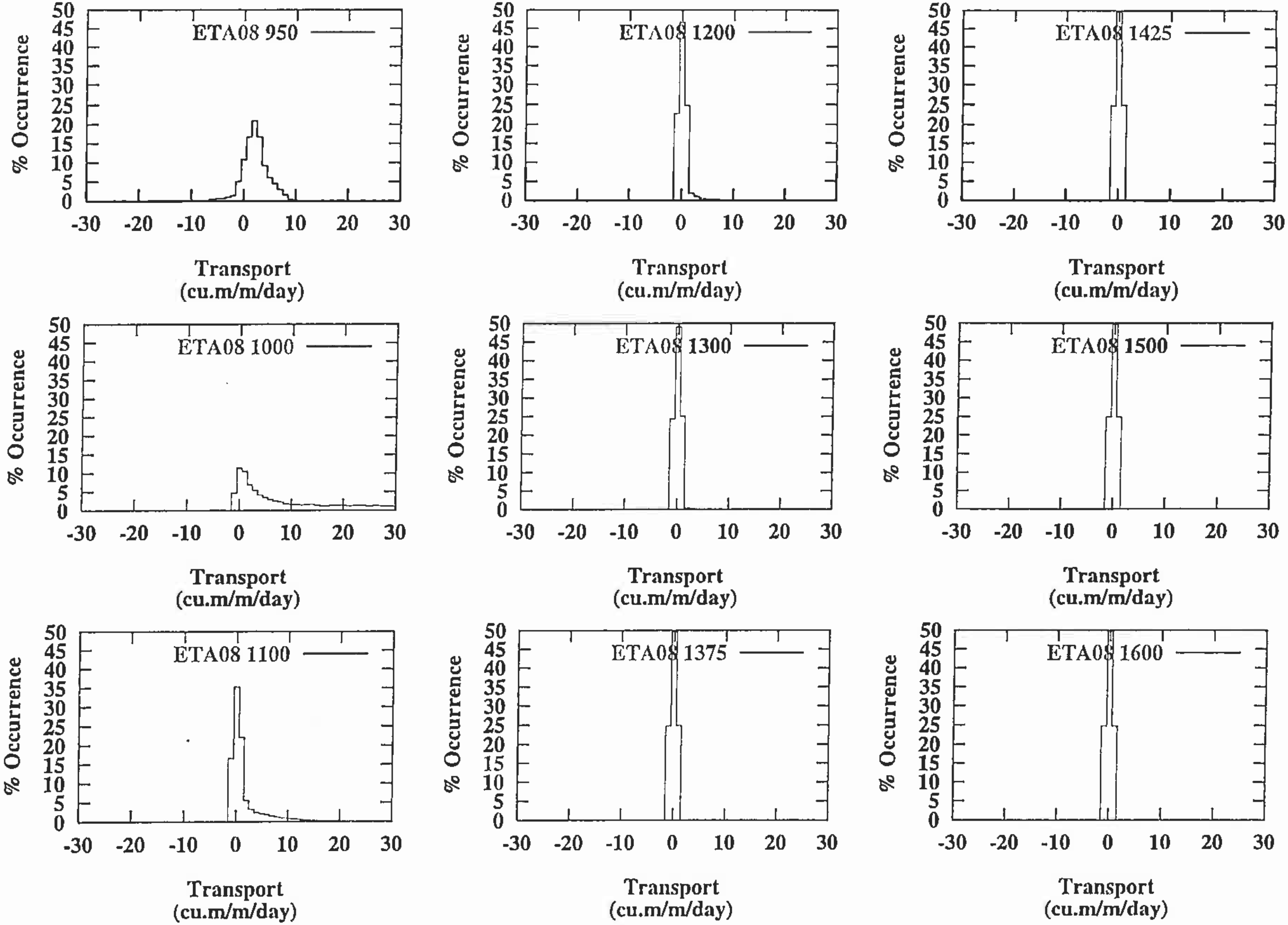
Longshore Sand Transport Probabilities Letitia Spit

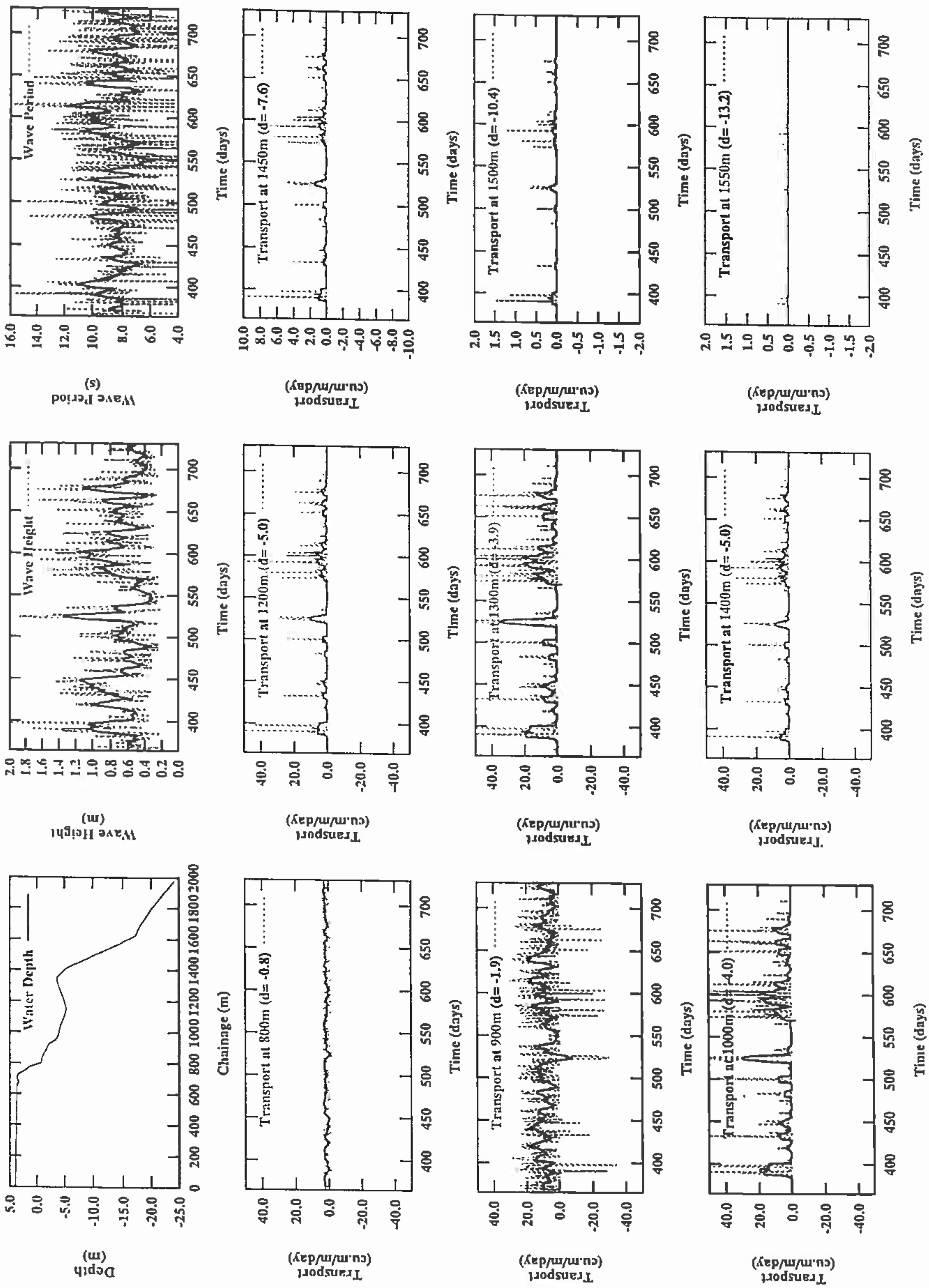
APPENDIX D: Cross-Shore Transport Results



Cross-shore transport - Line ETA 8

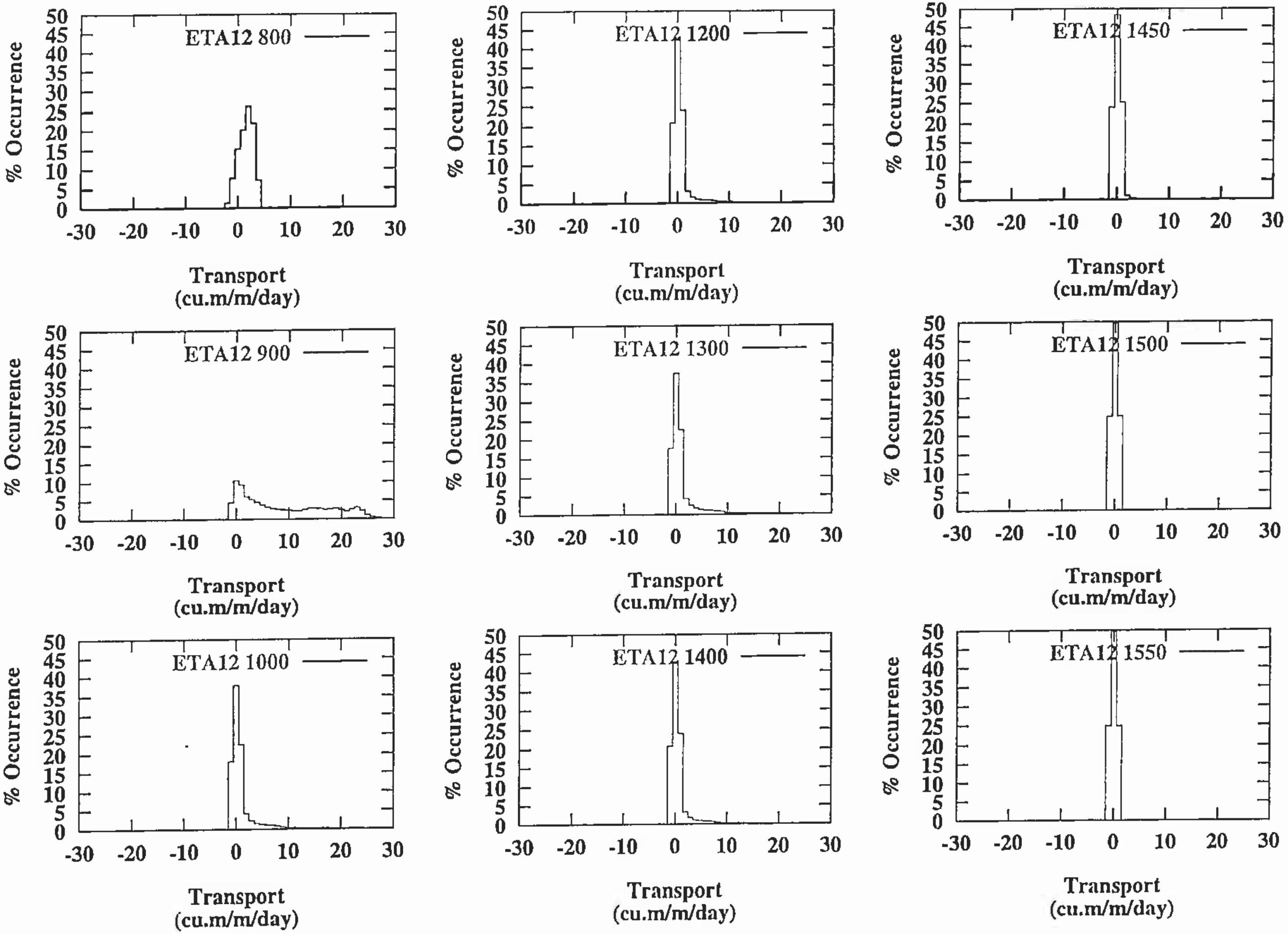
Cross-shore transport - Line ETA 8



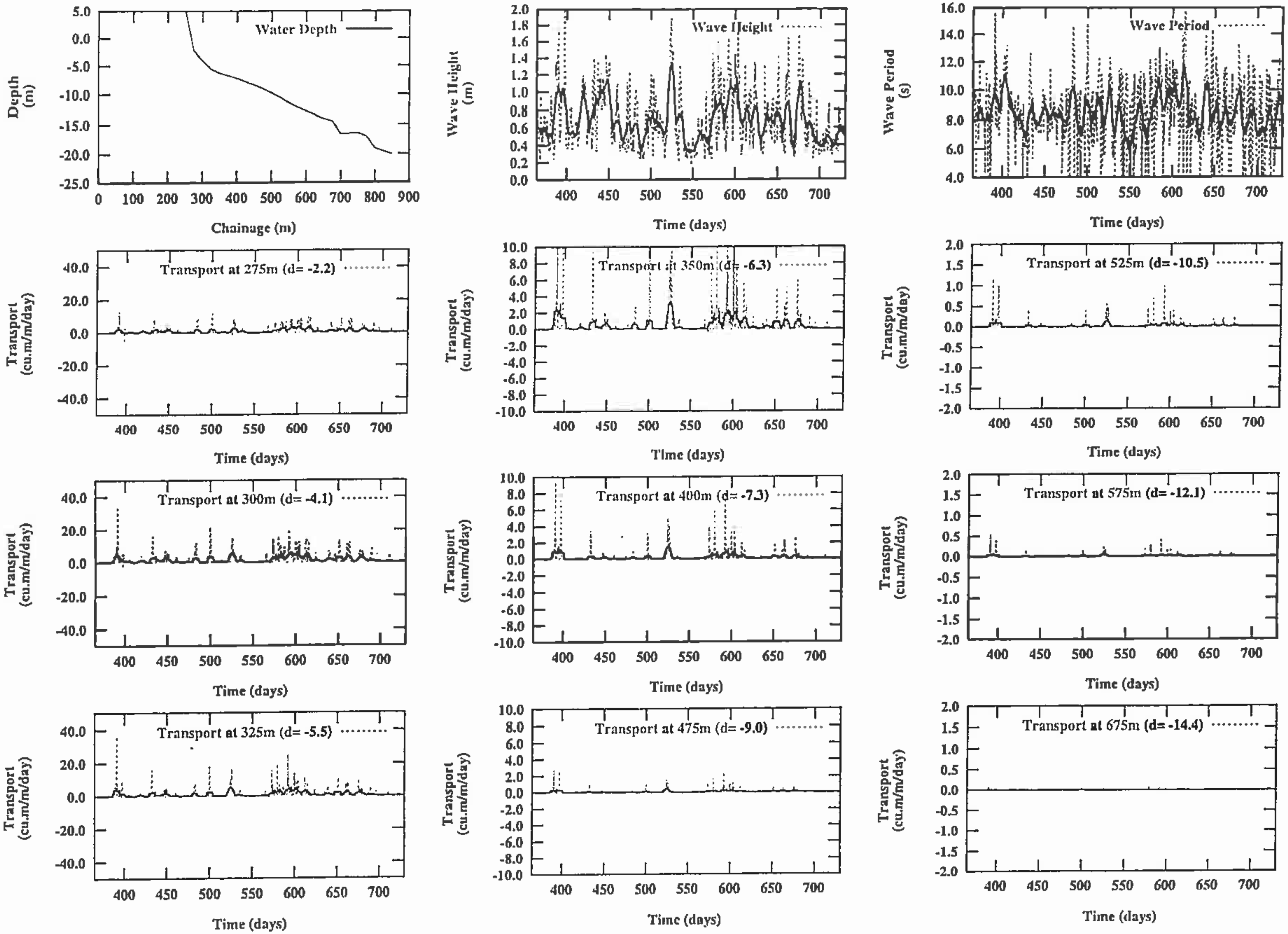


Cross-shore transport - Line ETA 12

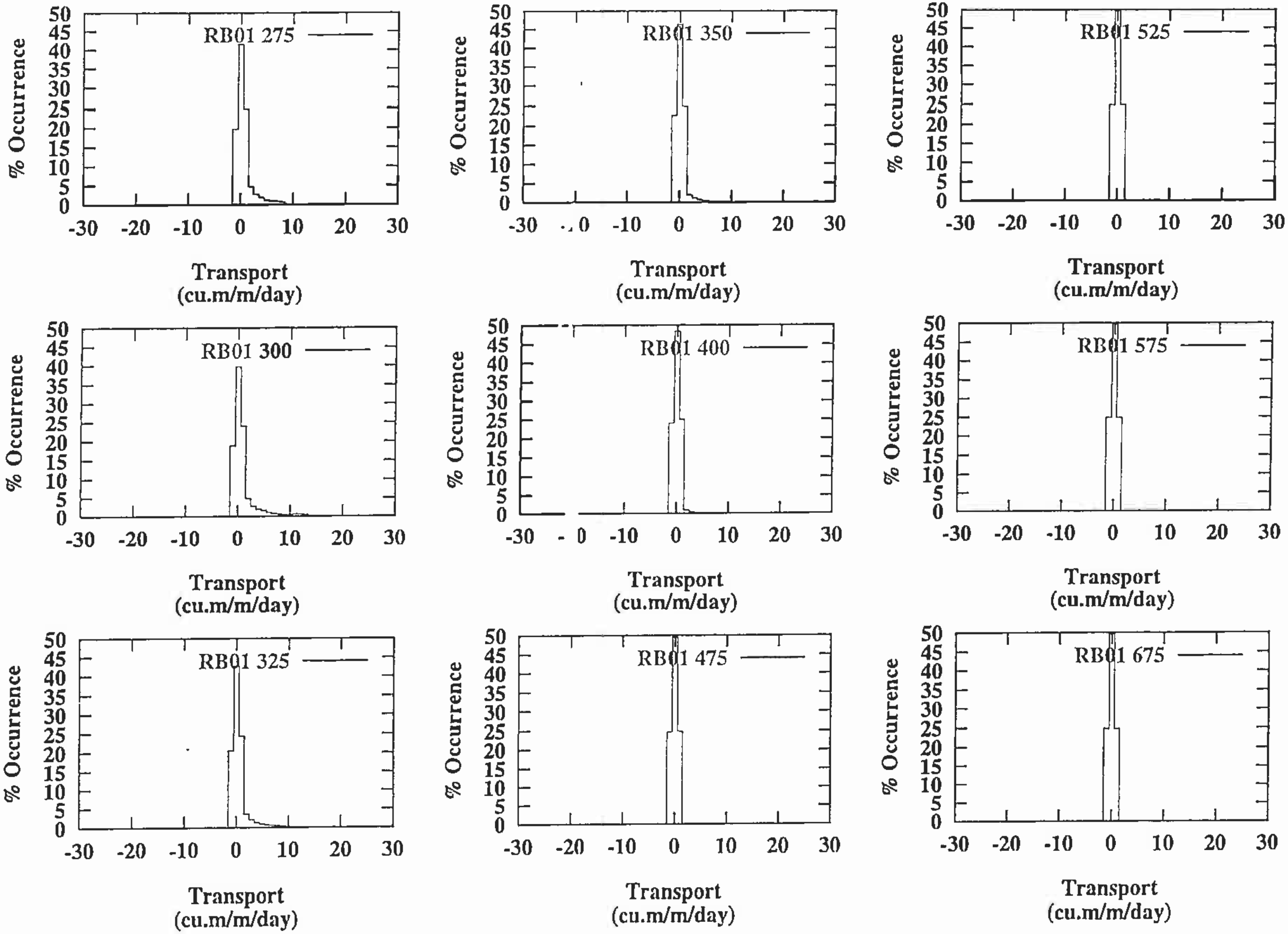
Cross-shore transport - Line ETA 12



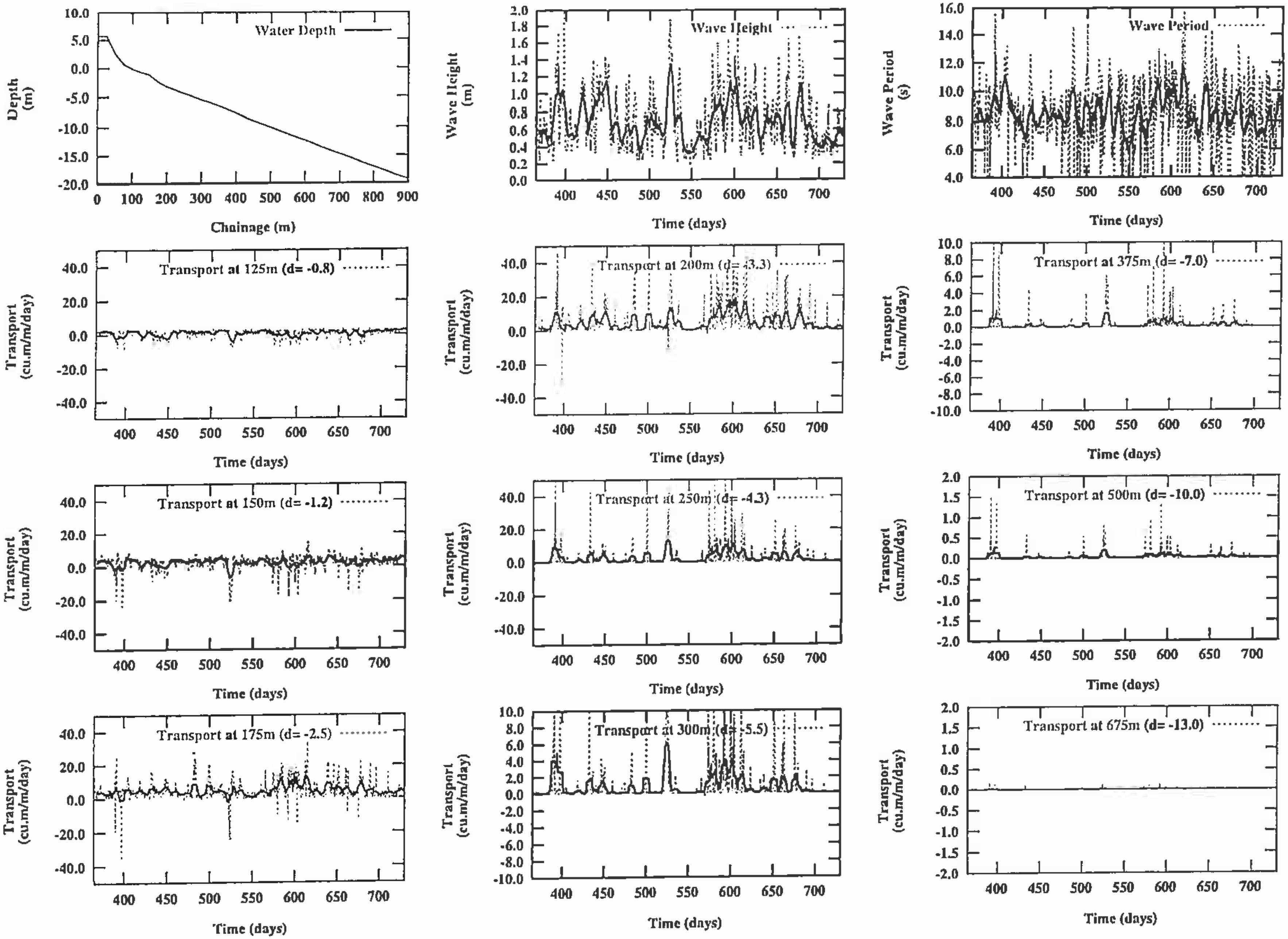
Cross-shore transport - Line RB1.0



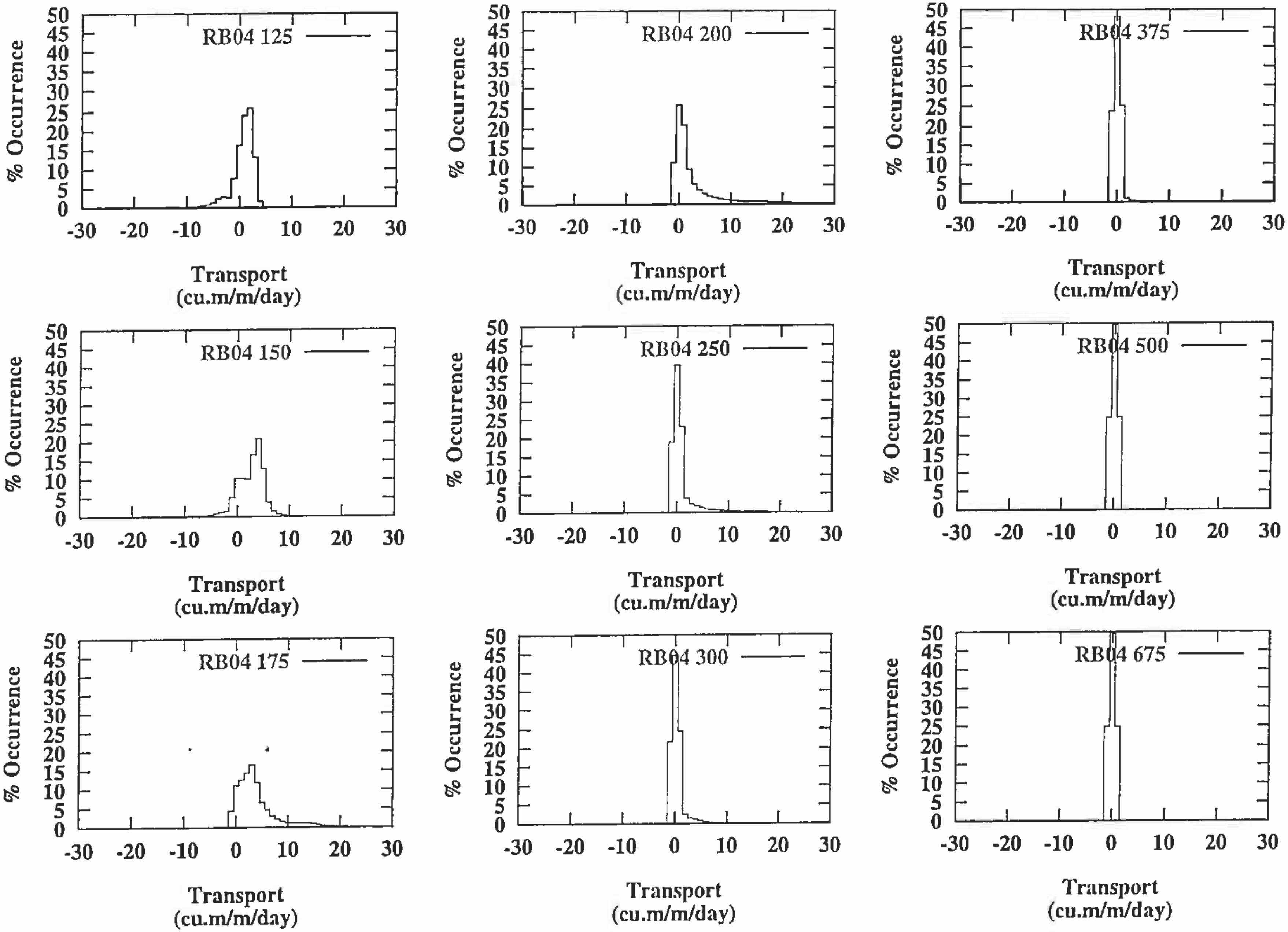
Cross-shore transport - Line RB1.0

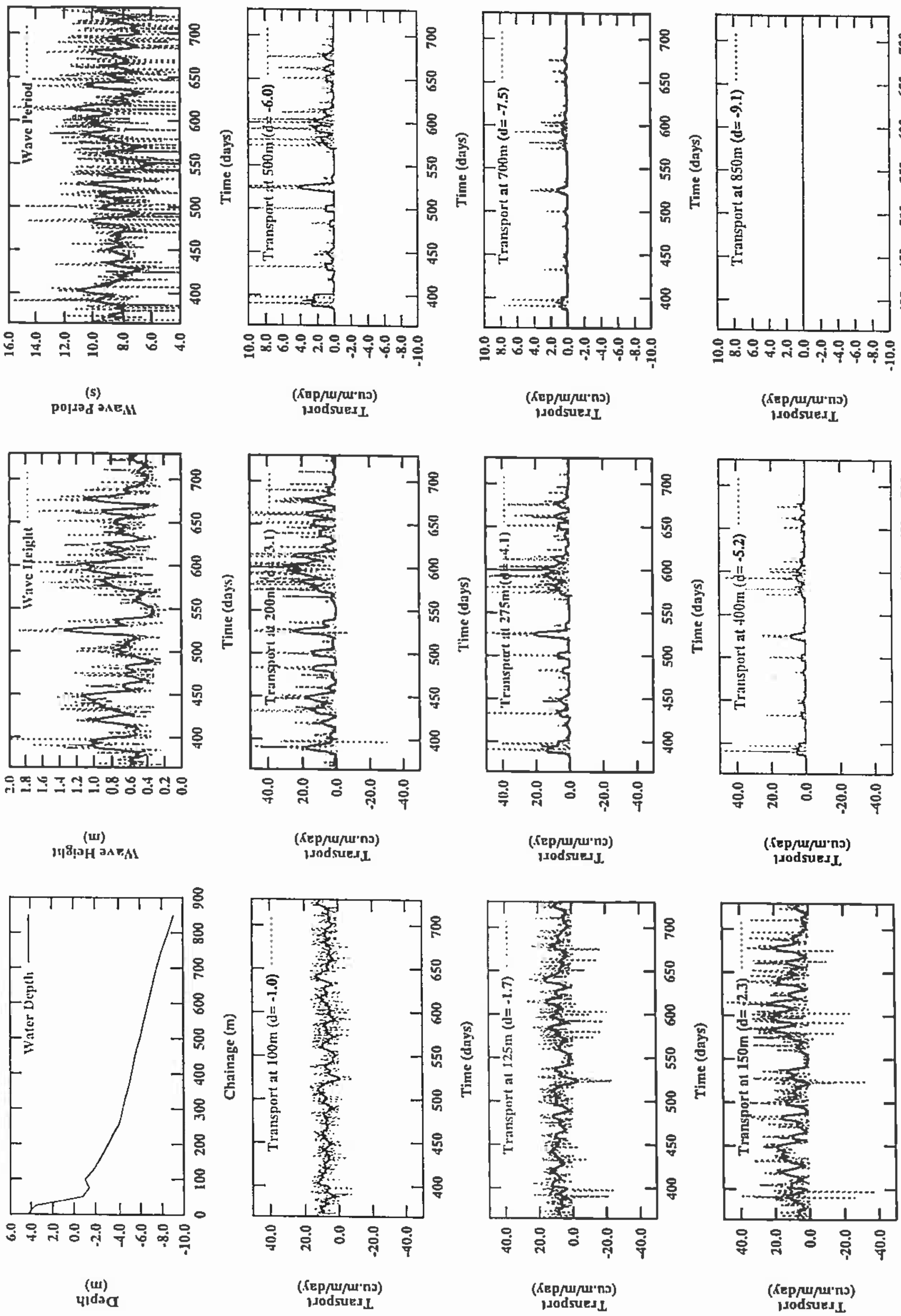


Cross-shore transport - Line RB4.0



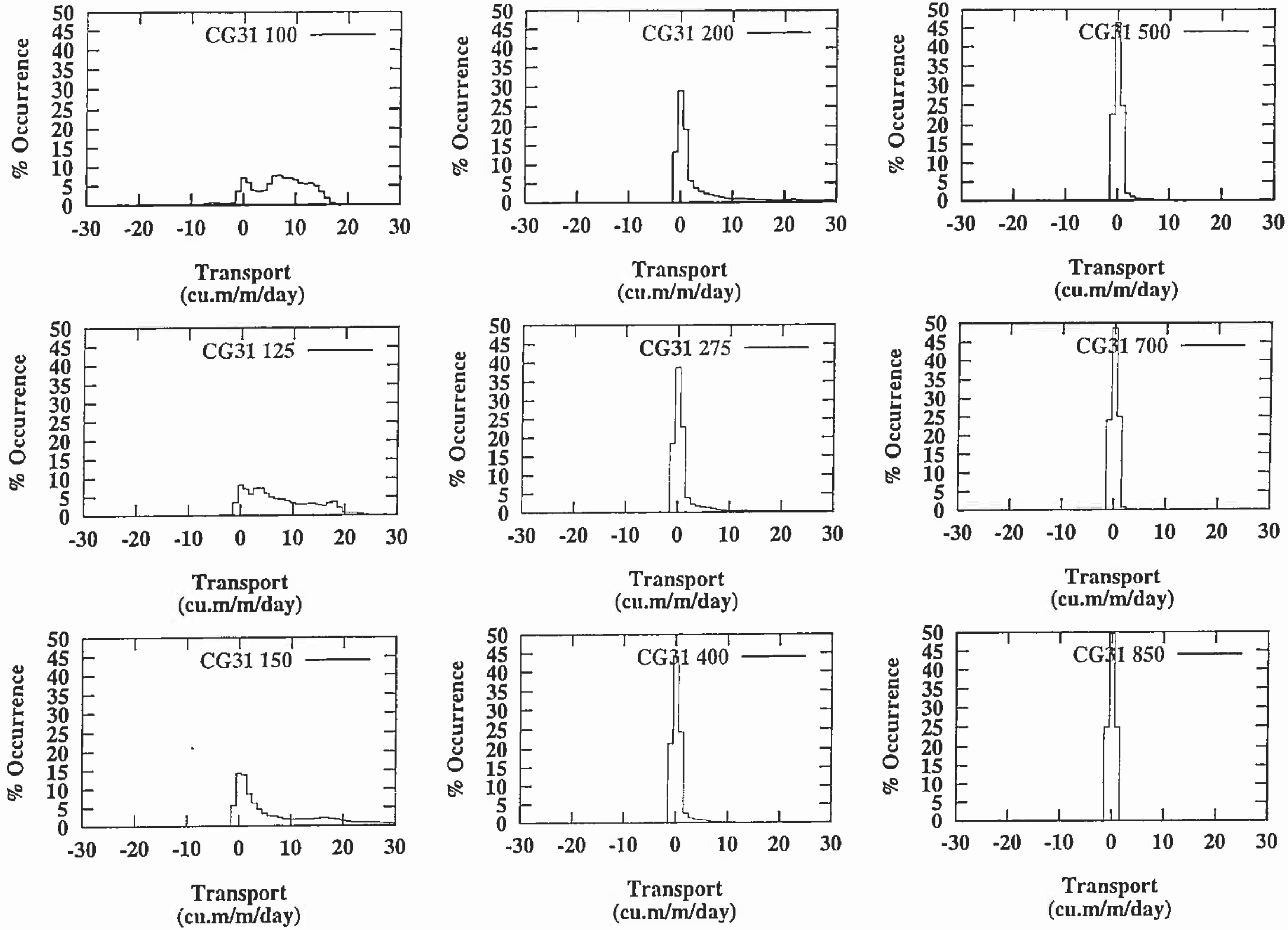
Cross-shore transport - Line RB4.0

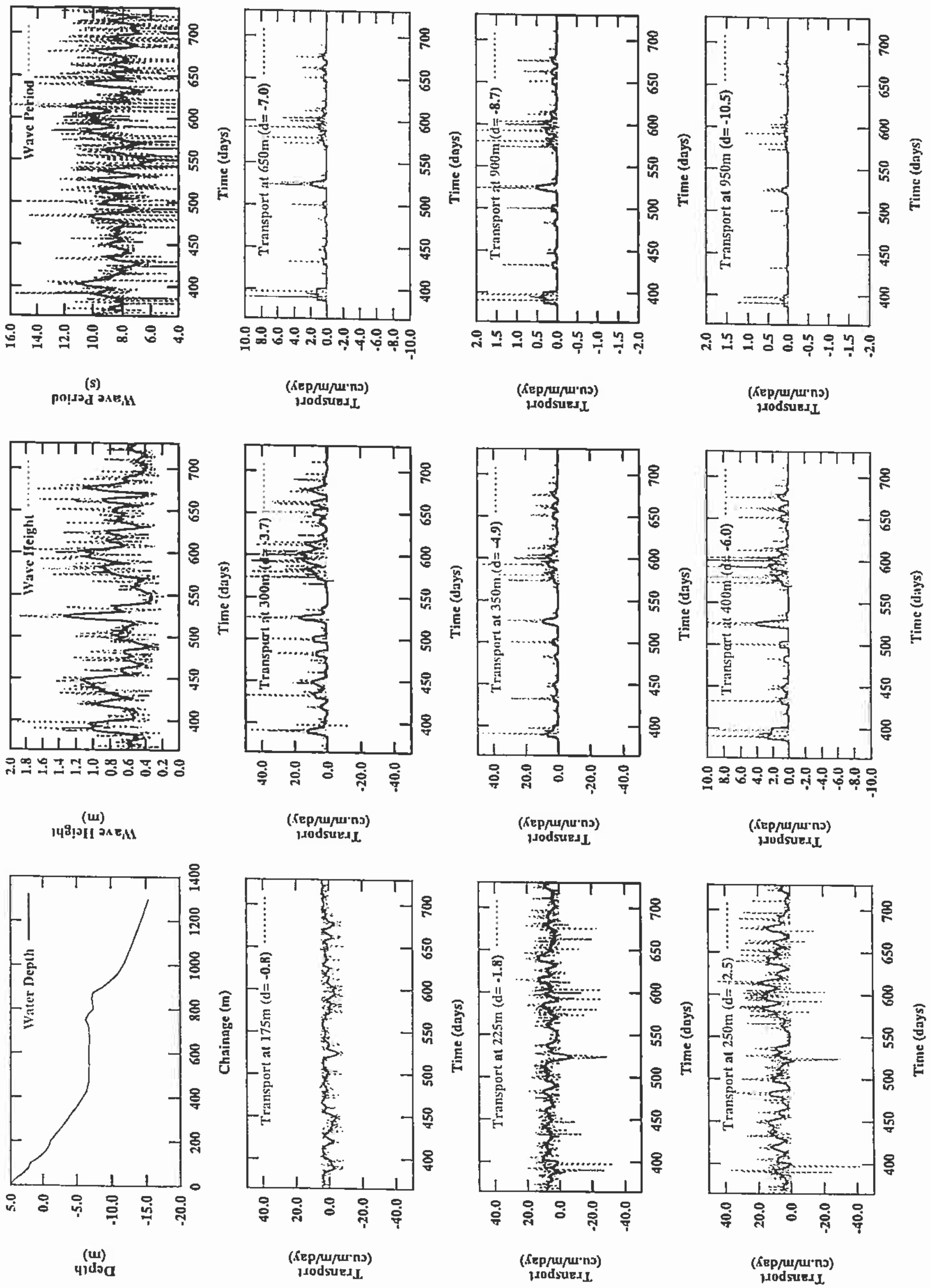




Cross-shore transport - Line CG3.1

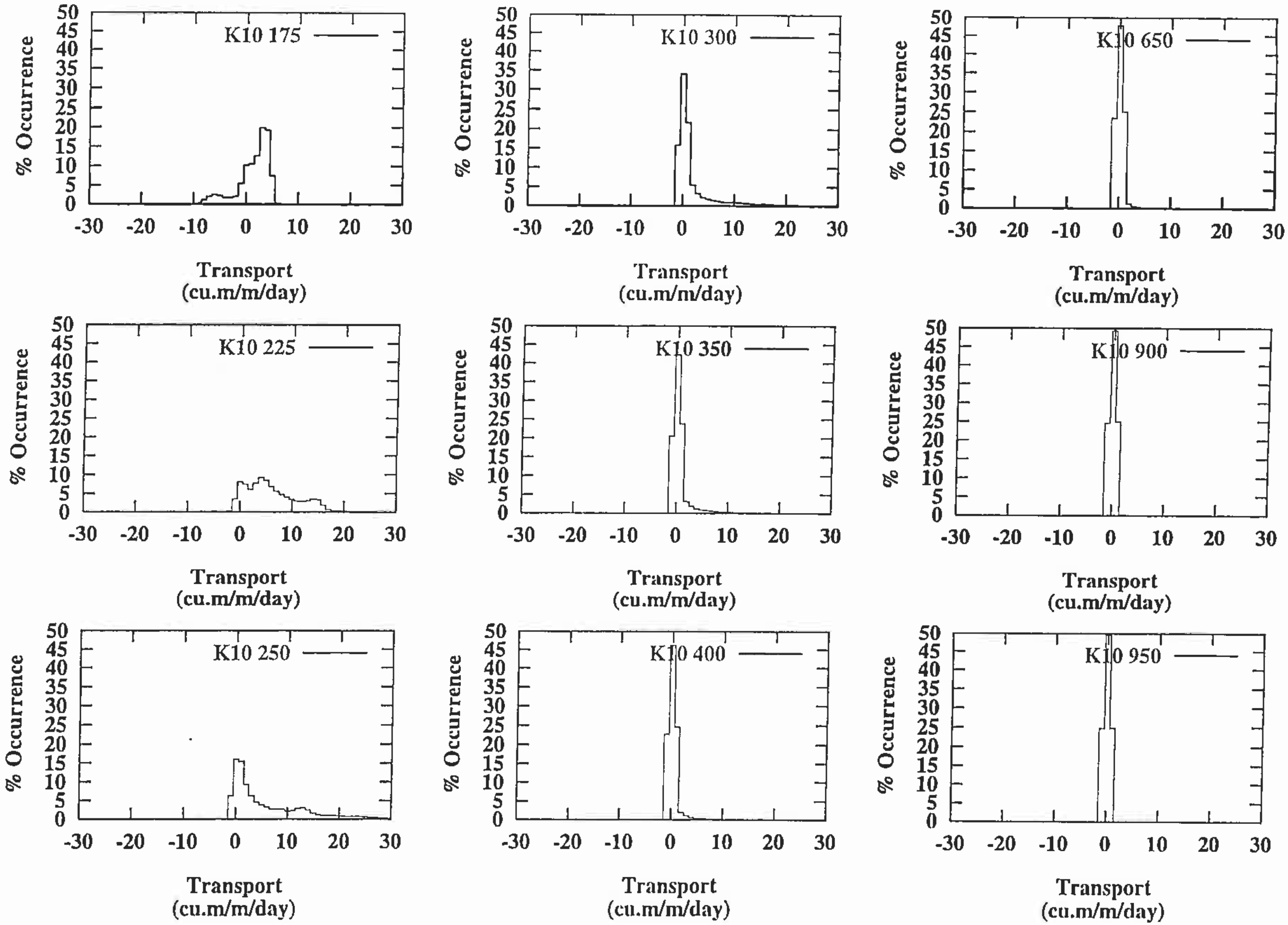
Cross-shore transport - Line CG3.1



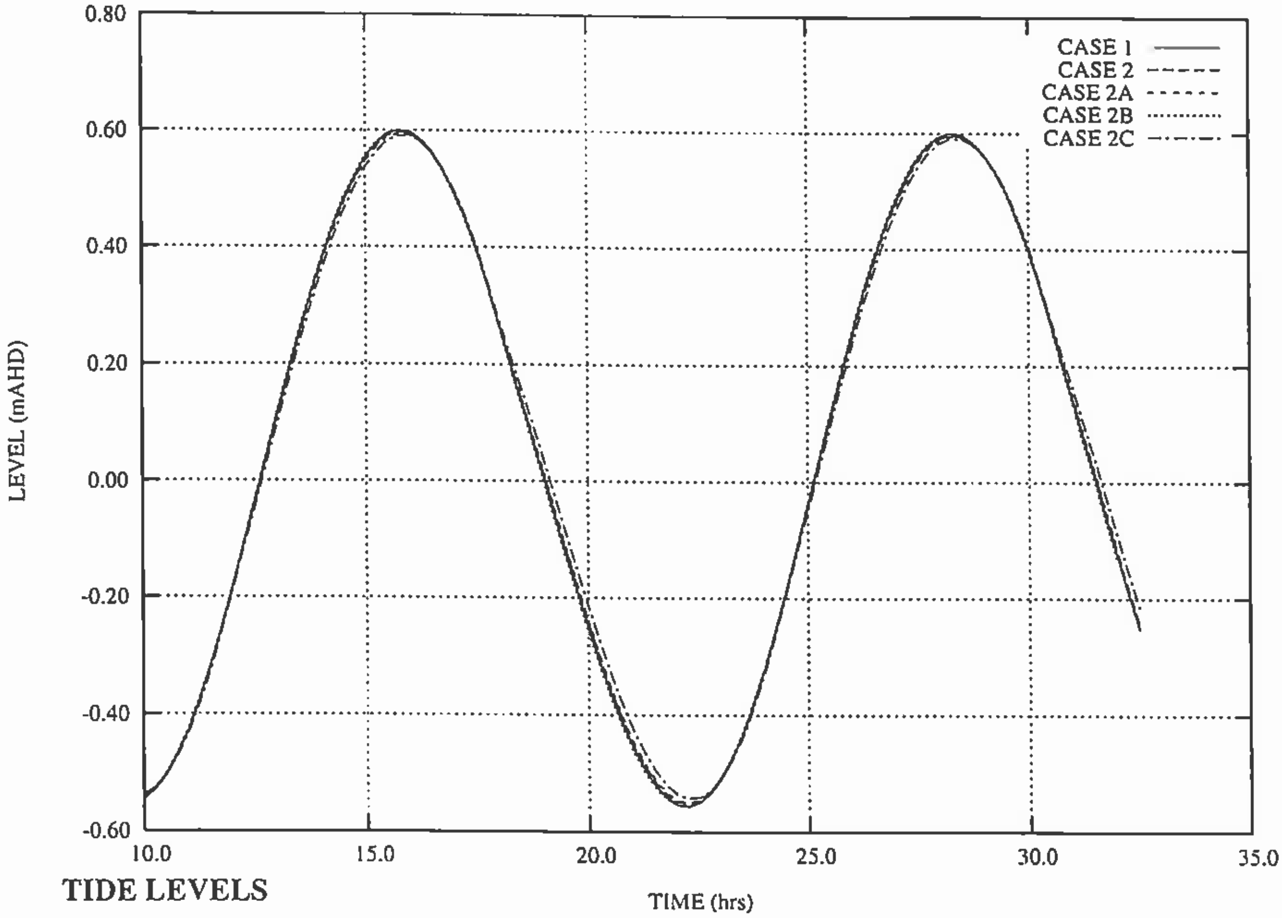


Cross-shore transport - Line K10.0

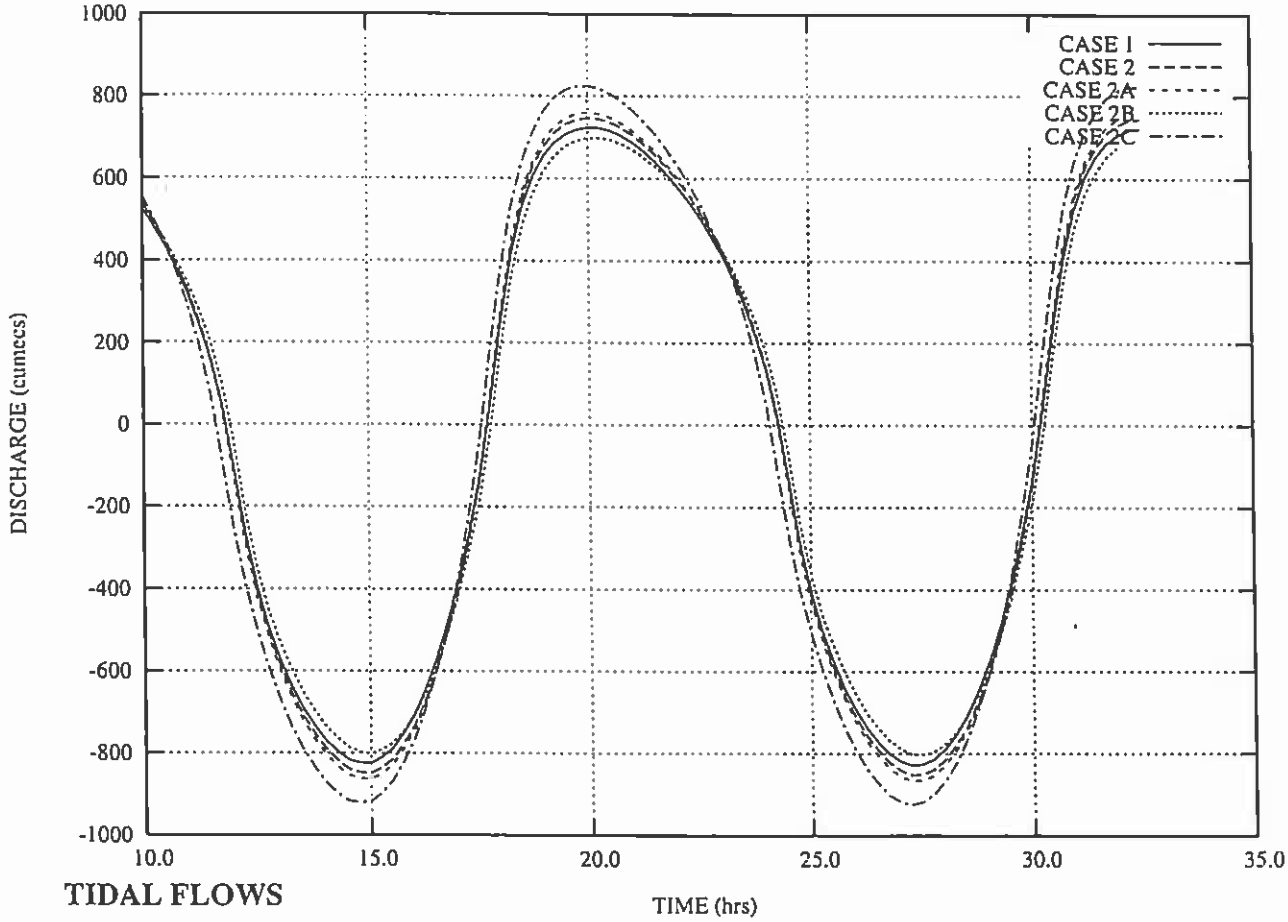
Cross-shore transport - Line K10.0



APPENDIX E: Tweed River Estuary Tidal Regime Impacts



TIDE LEVELS

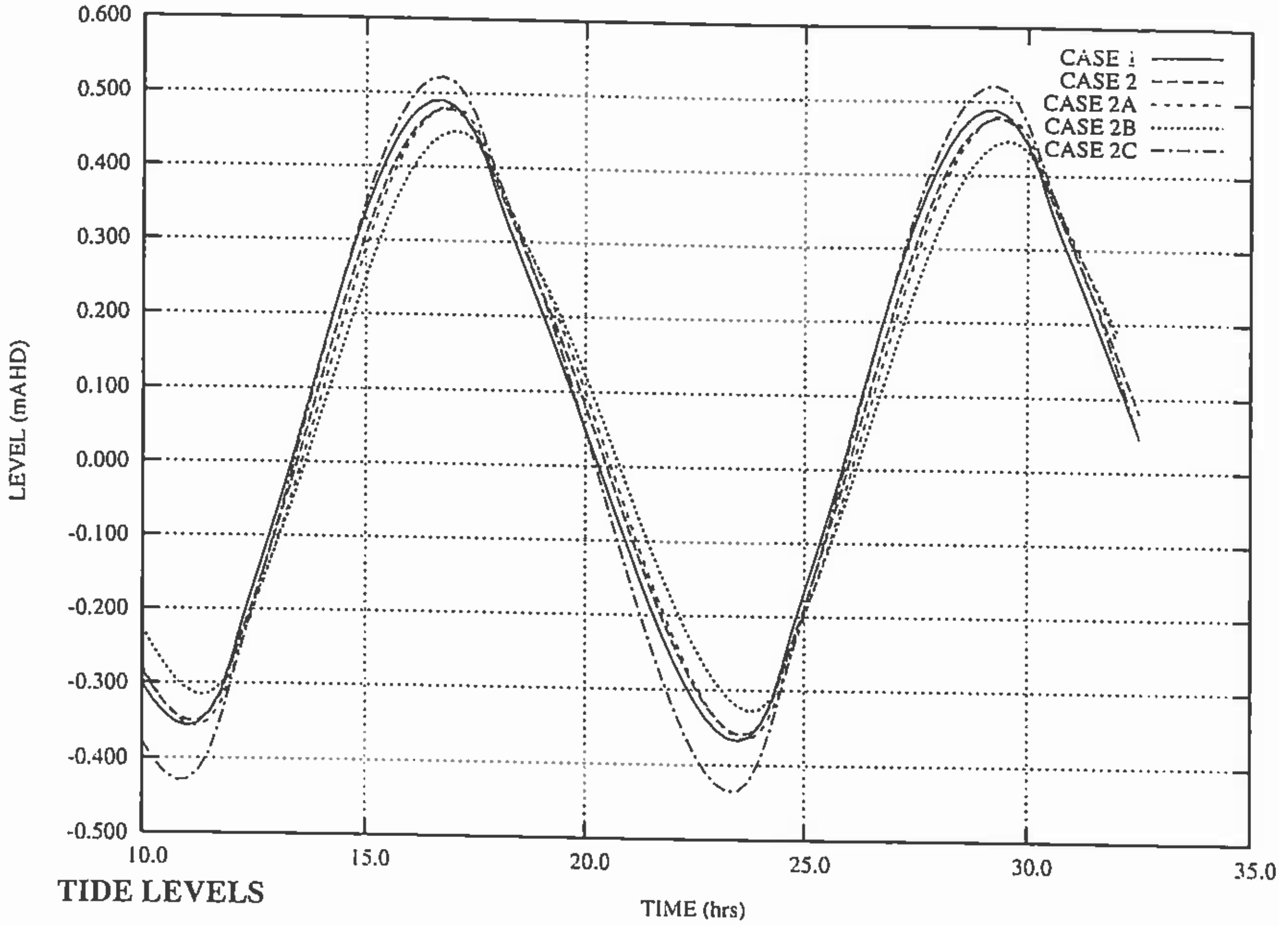


TIDAL FLOWS

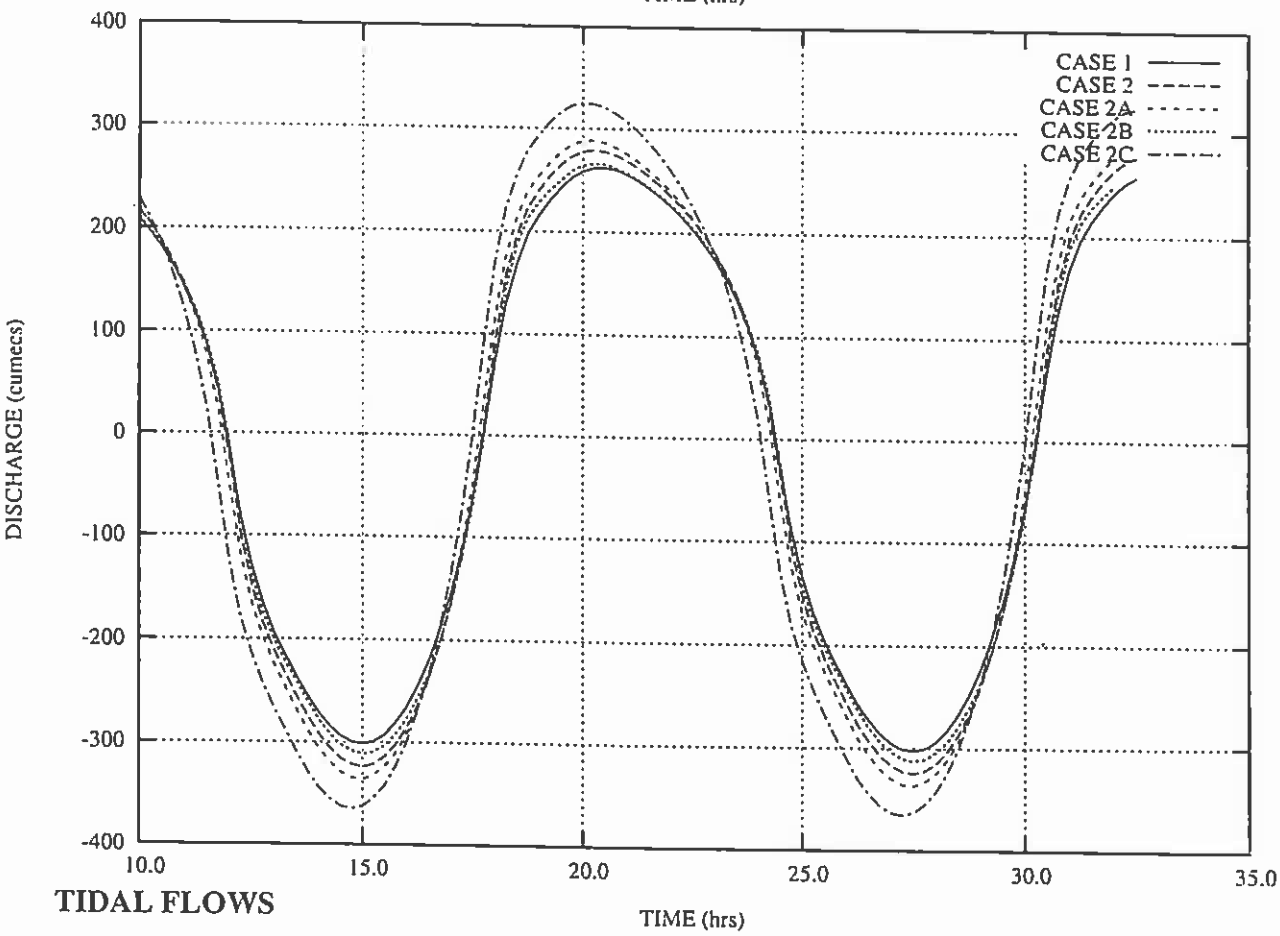
REGIONAL GAUGE - EXISTING SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

Figure E.1

WBM



TIDE LEVELS

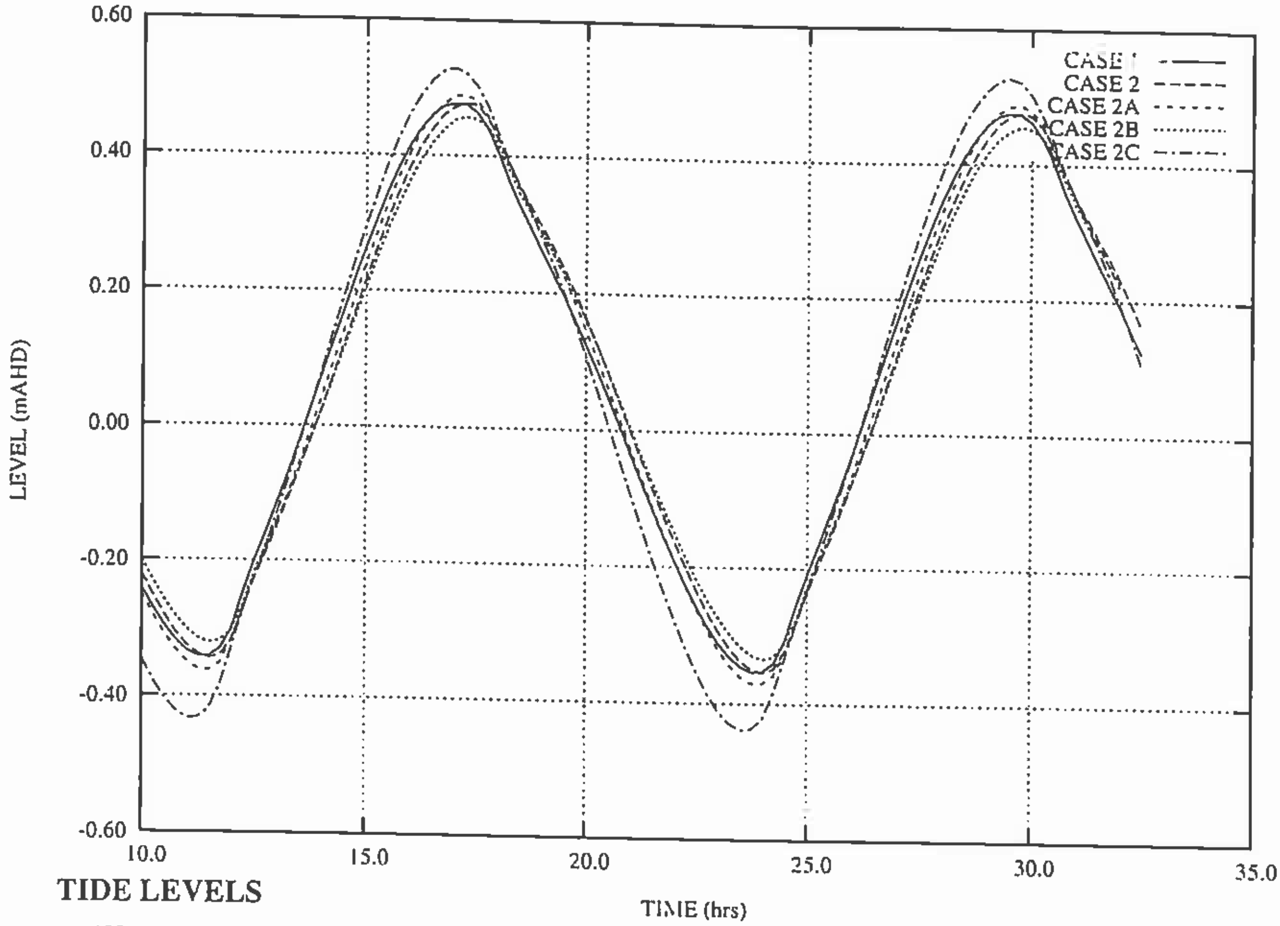


TIDAL FLOWS

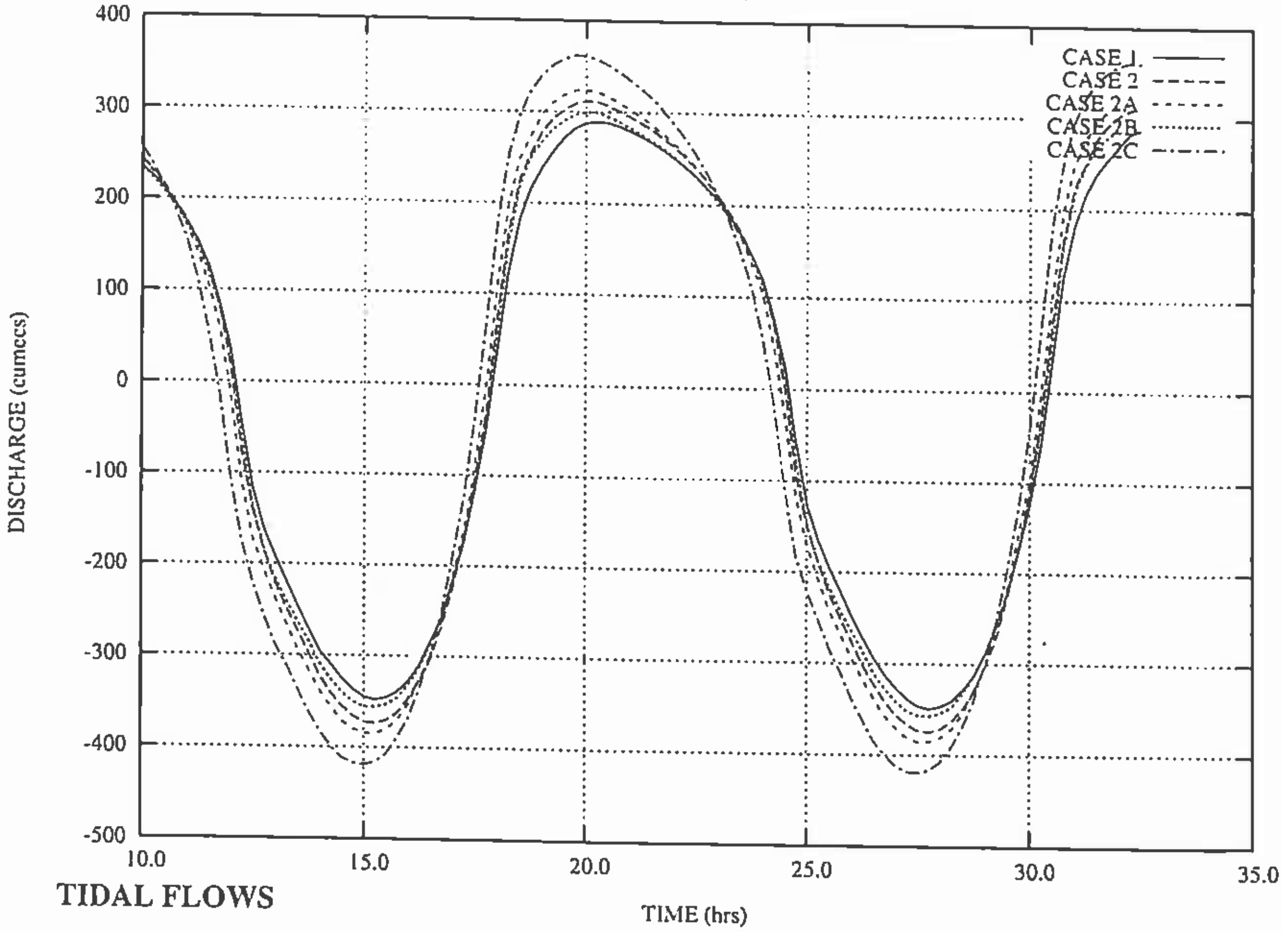
LETITIA 2B - EXISTING SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

Figure
E.3

WBM



TIDE LEVELS

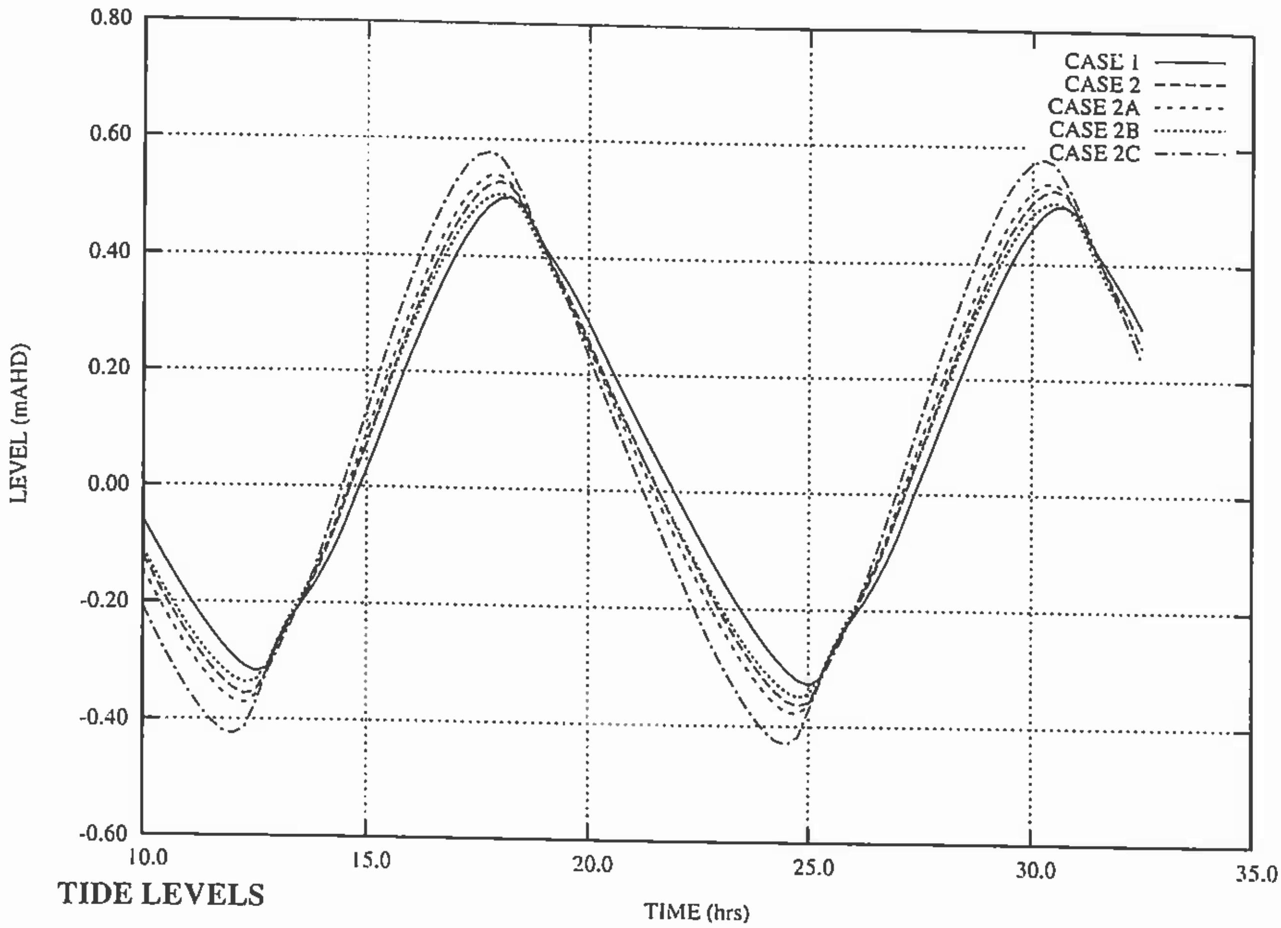


TIDAL FLOWS

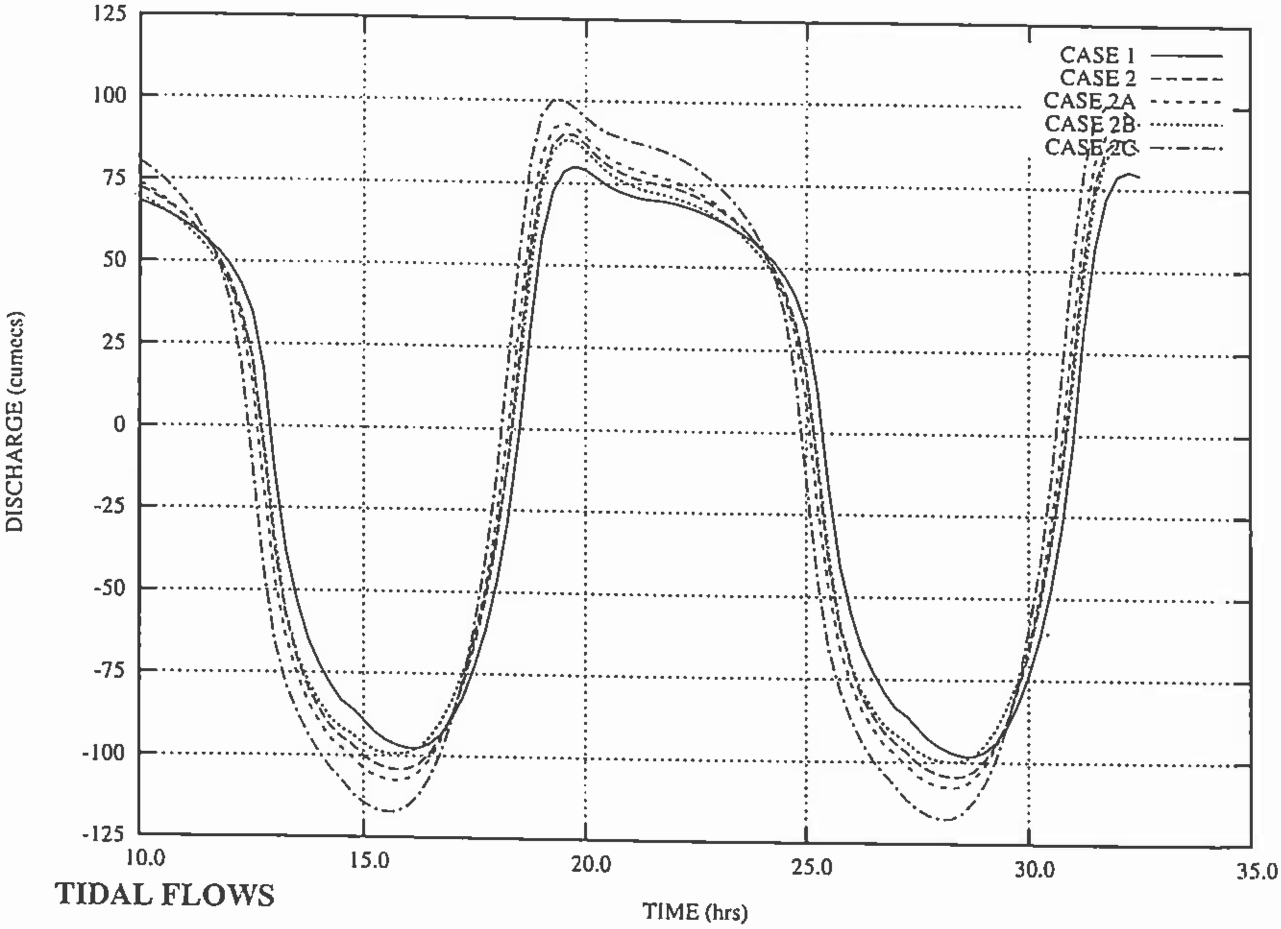
BARNEYS POINT - EXISTING SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

Figure E.4

WBM



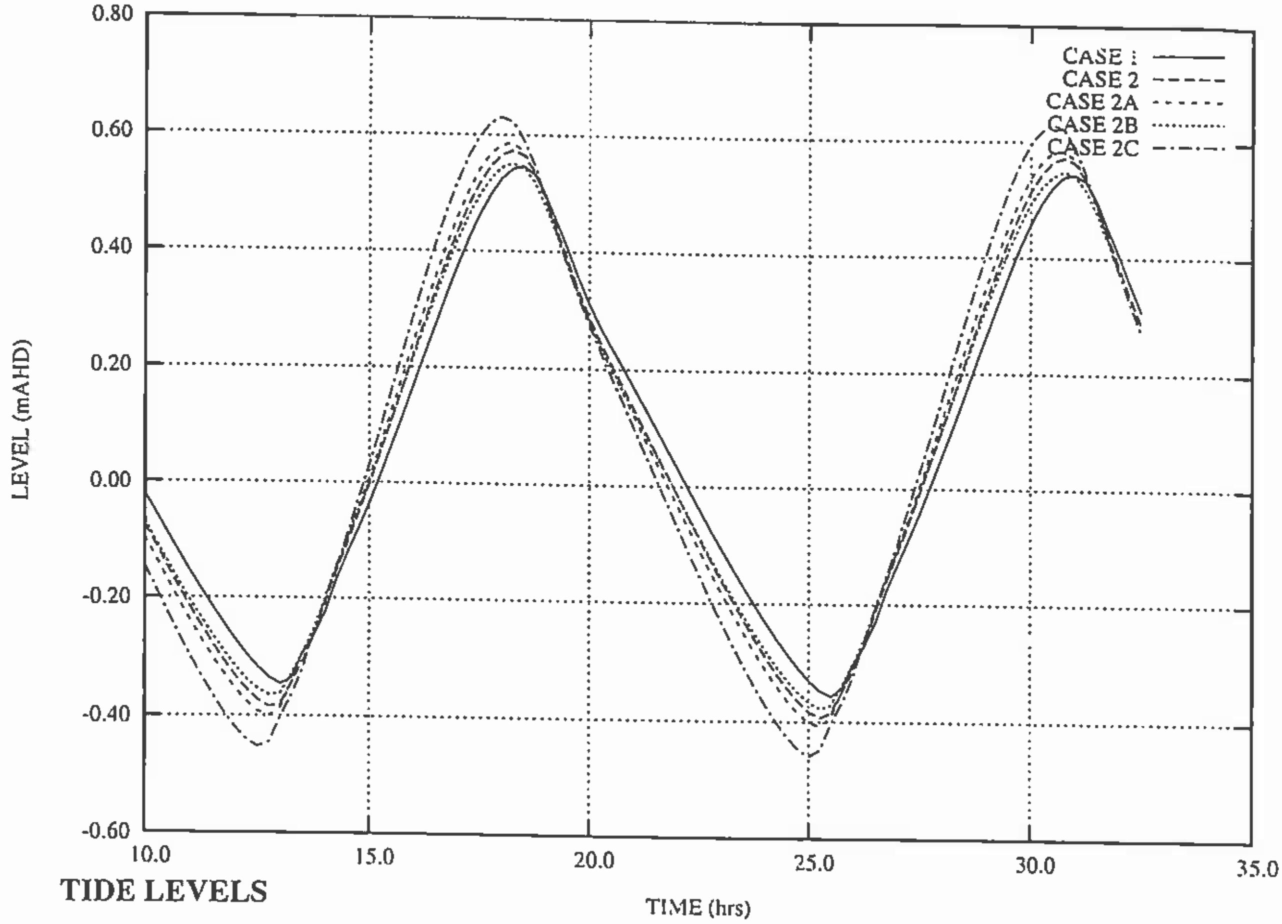
TIDE LEVELS



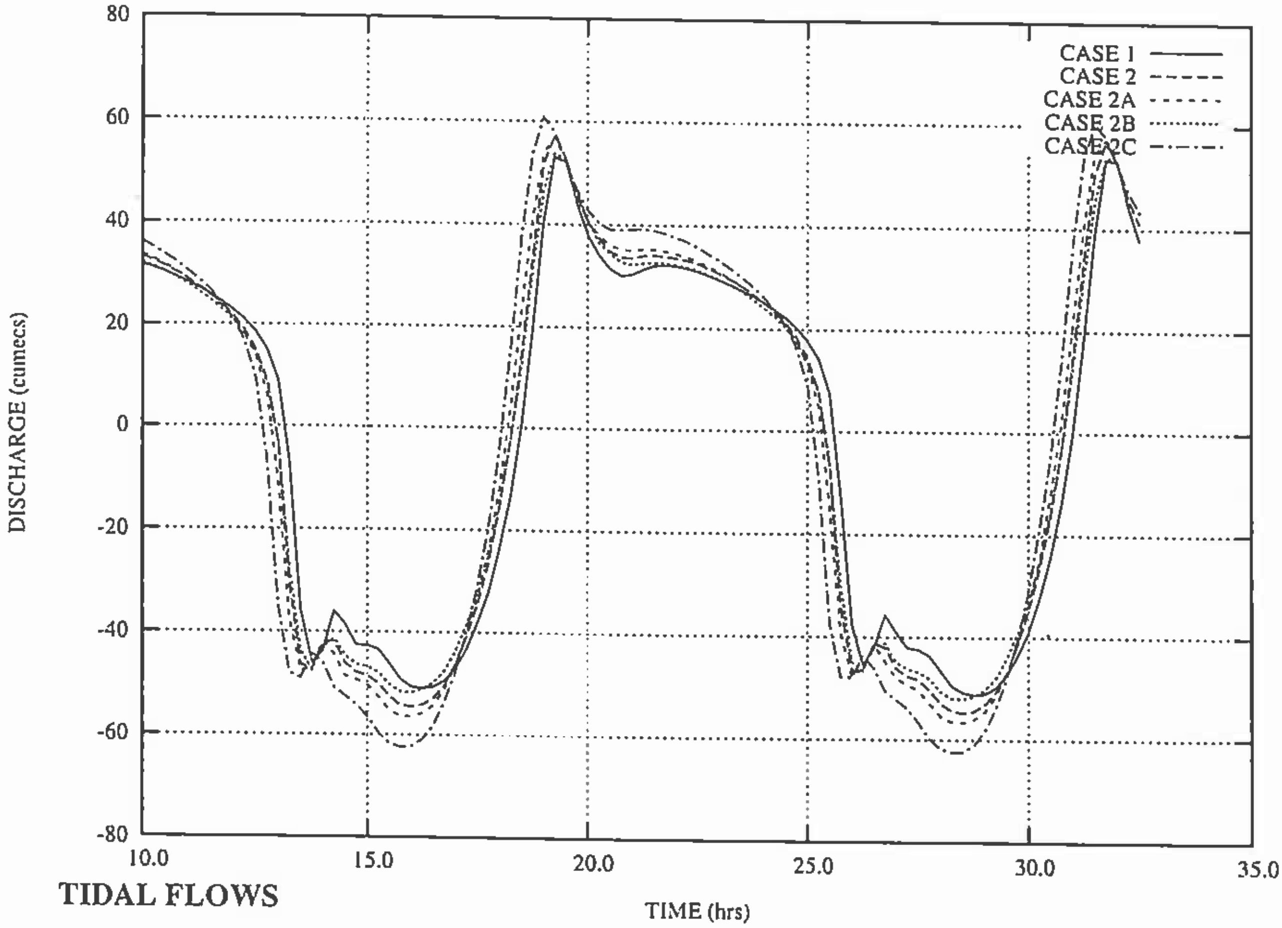
TIDAL FLOWS

TUMBULGUM - EXISTING SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

Figure E.5



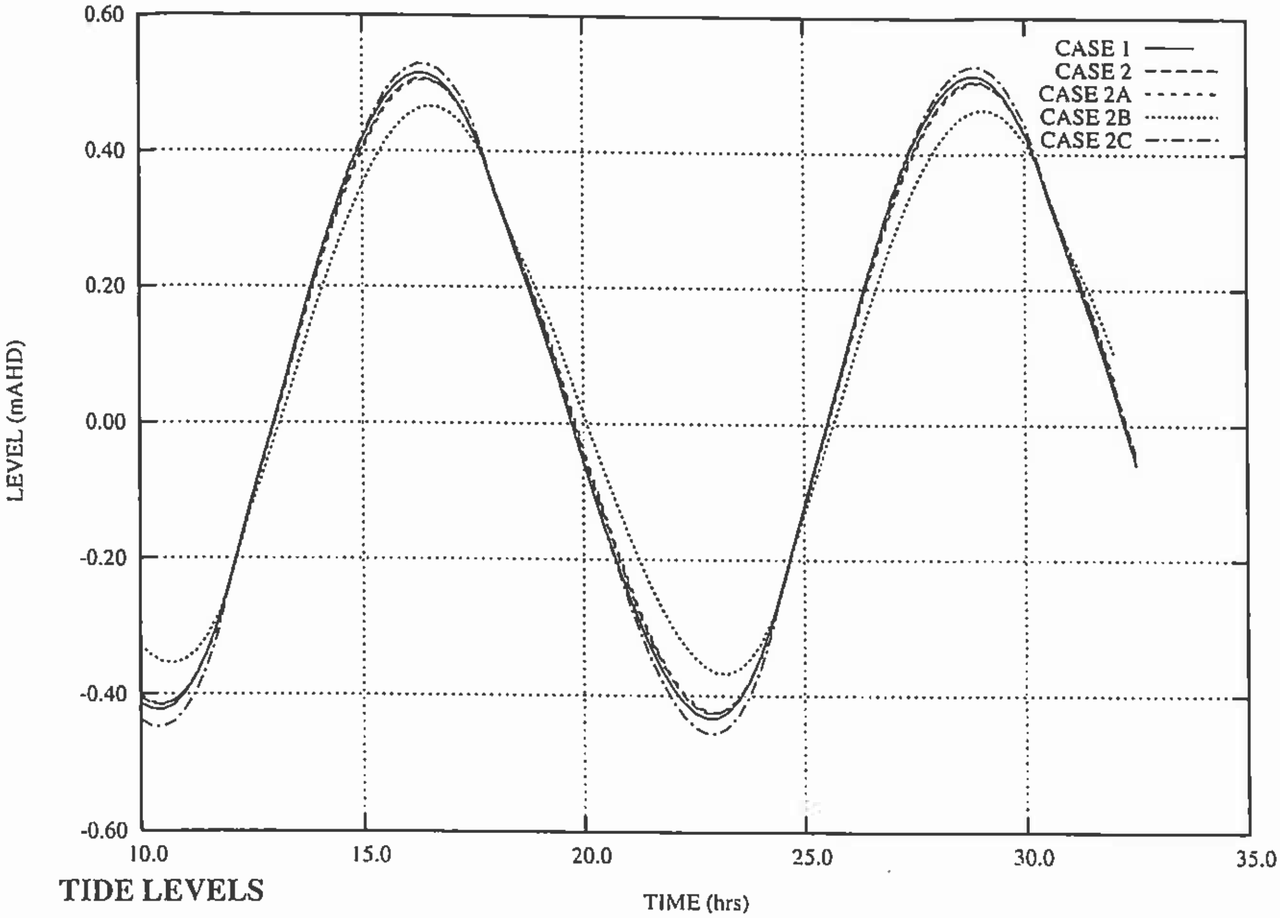
TIDE LEVELS



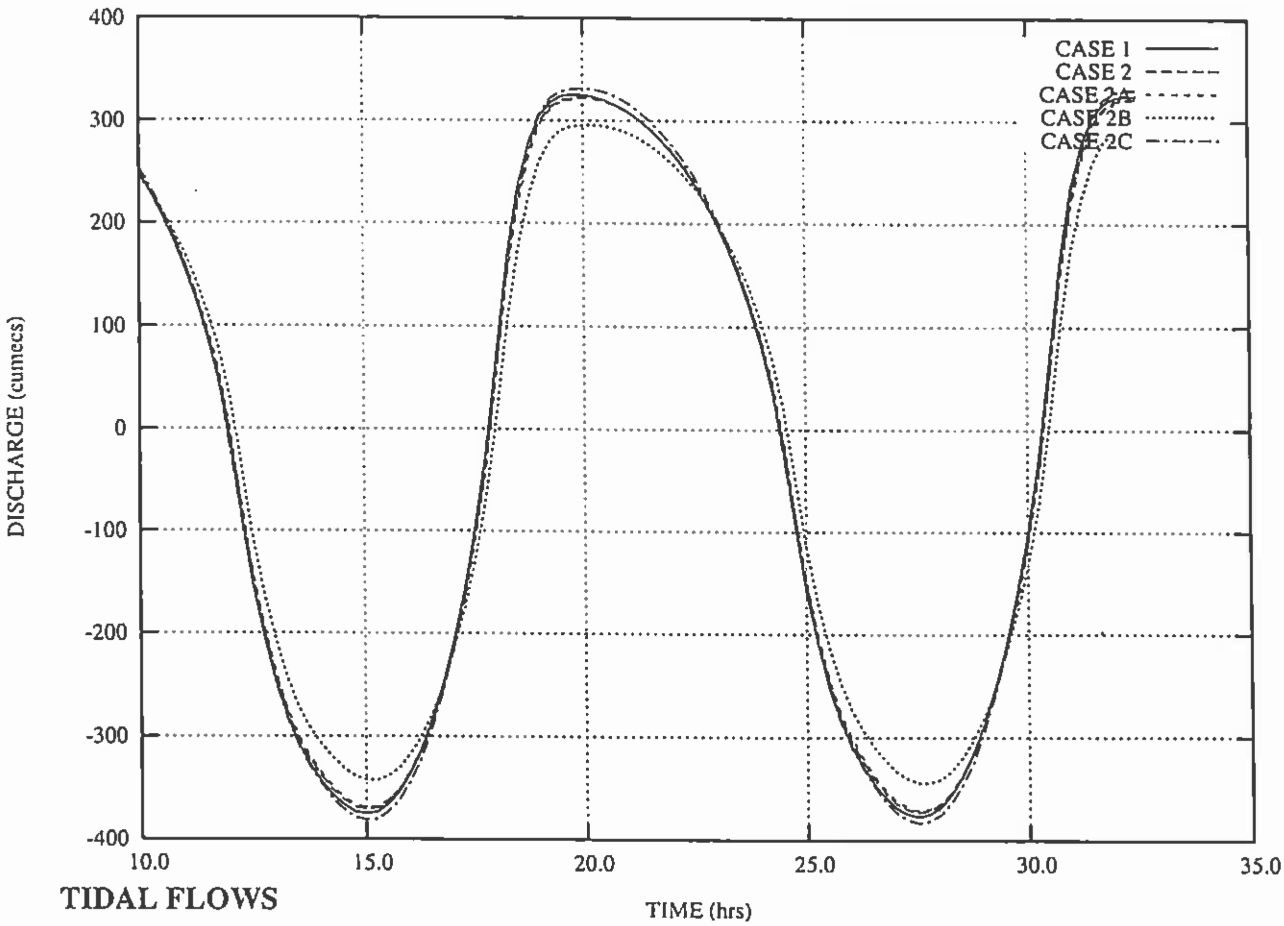
TIDAL FLOWS

MURWILLUMBAH - EXISTING SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

Figure E.6



TIDE LEVELS

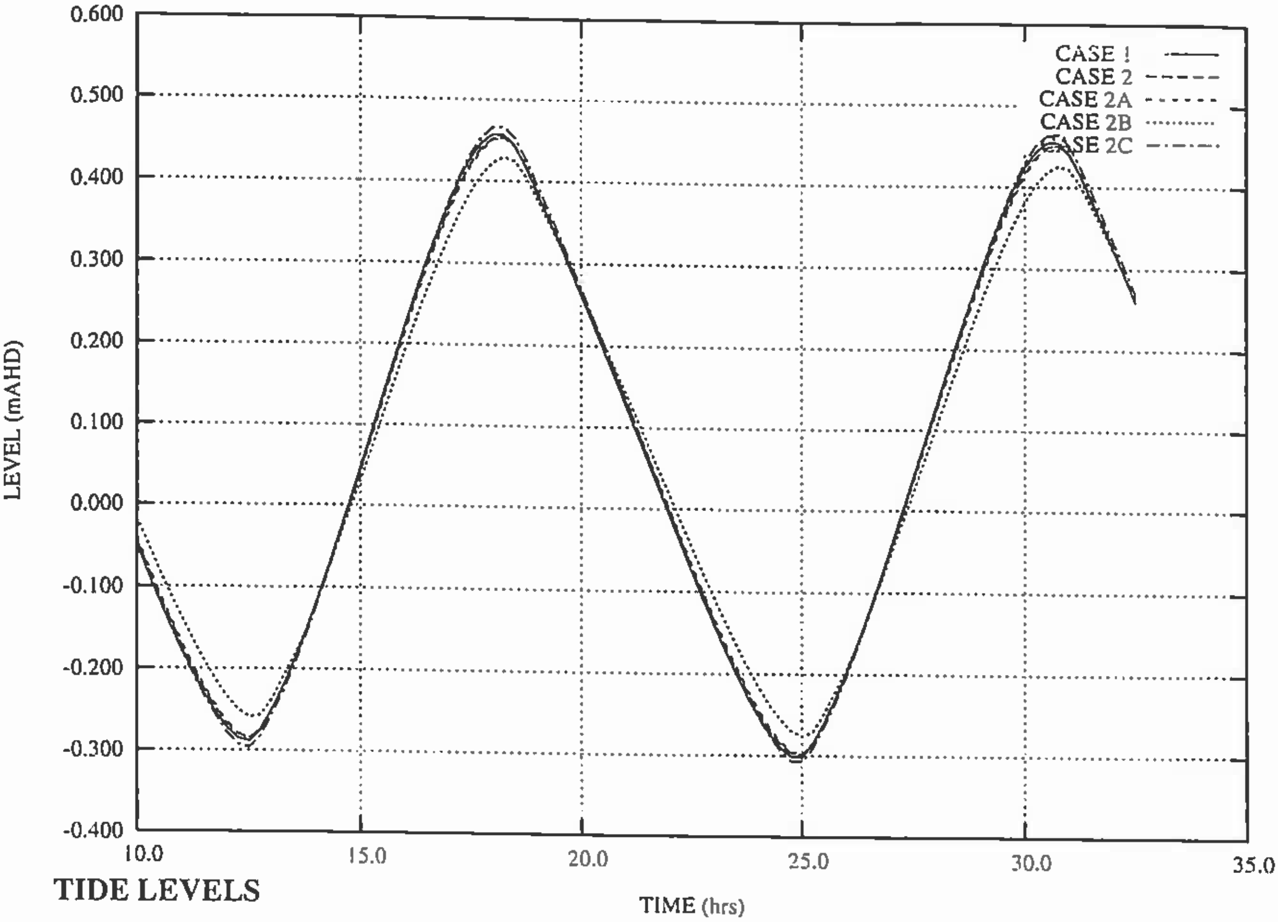


TIDAL FLOWS

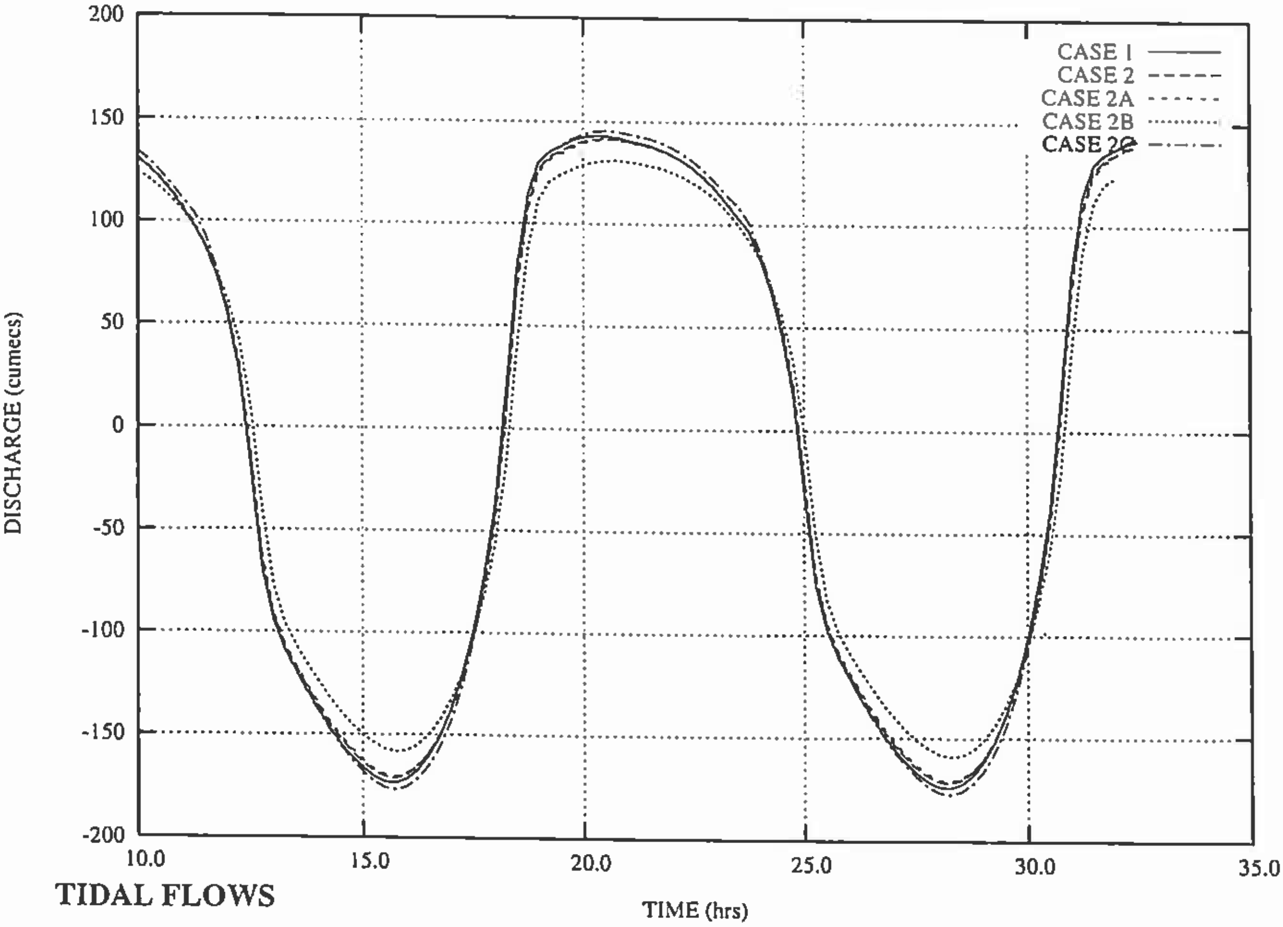
TERRANORA INLET - EXISTING SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

Figure E.7

WBM



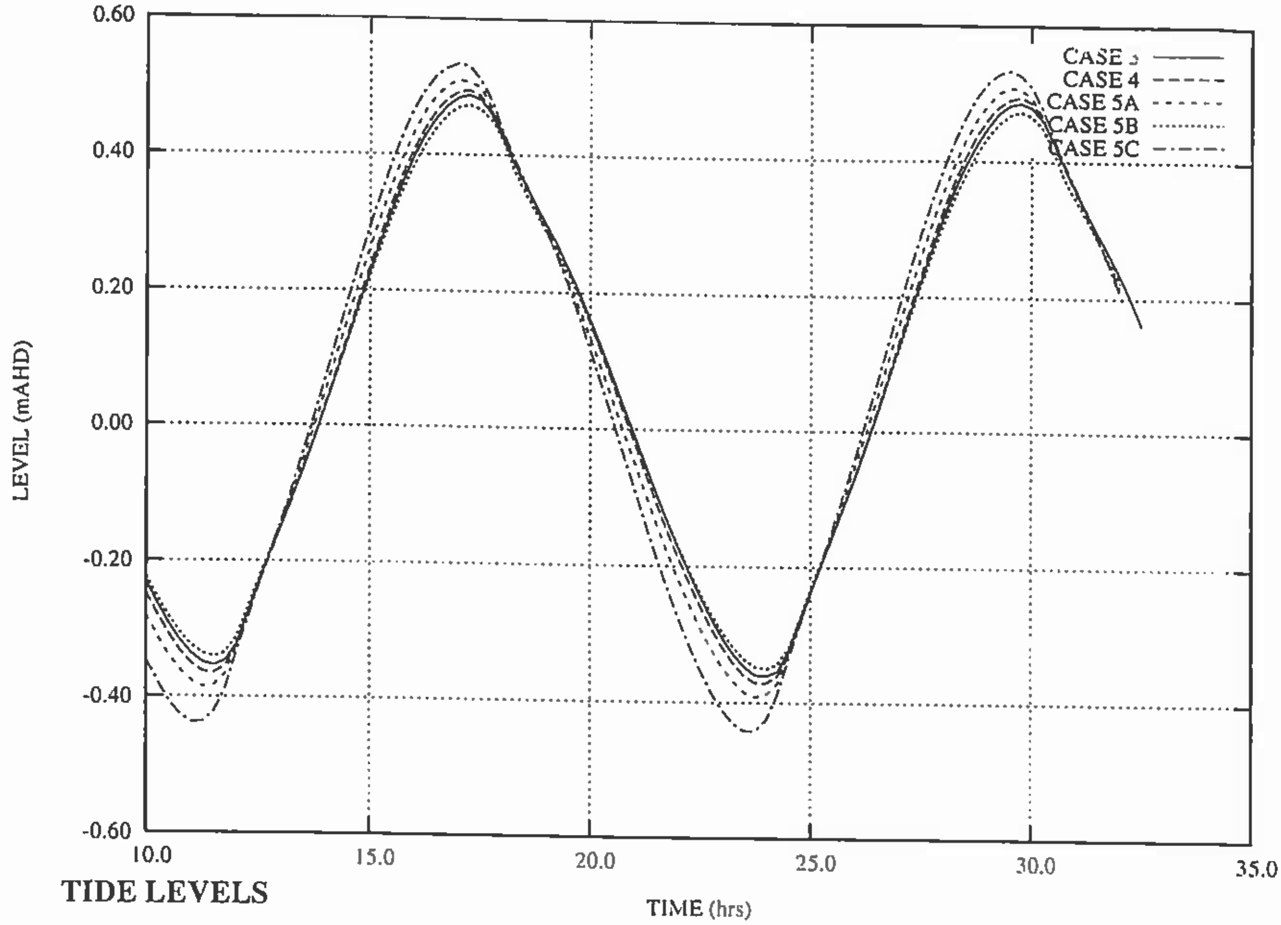
TIDE LEVELS



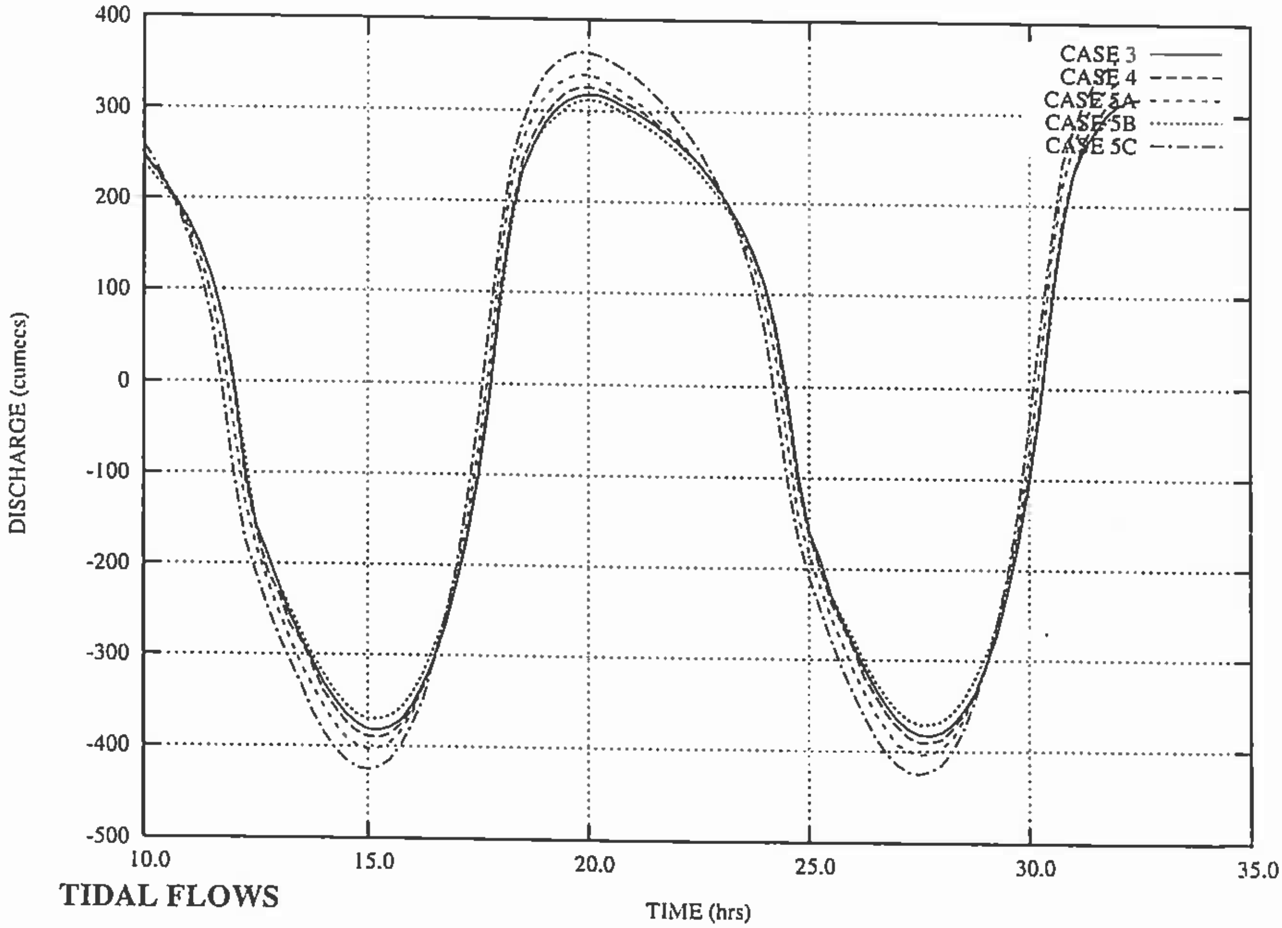
TIDAL FLOWS

TERRANORA BROADWATER - EXISTINGE SCENARIOS
MEAN SPRING TIDE : LEVELS AND FLOWS

F



TIDE LEVELS

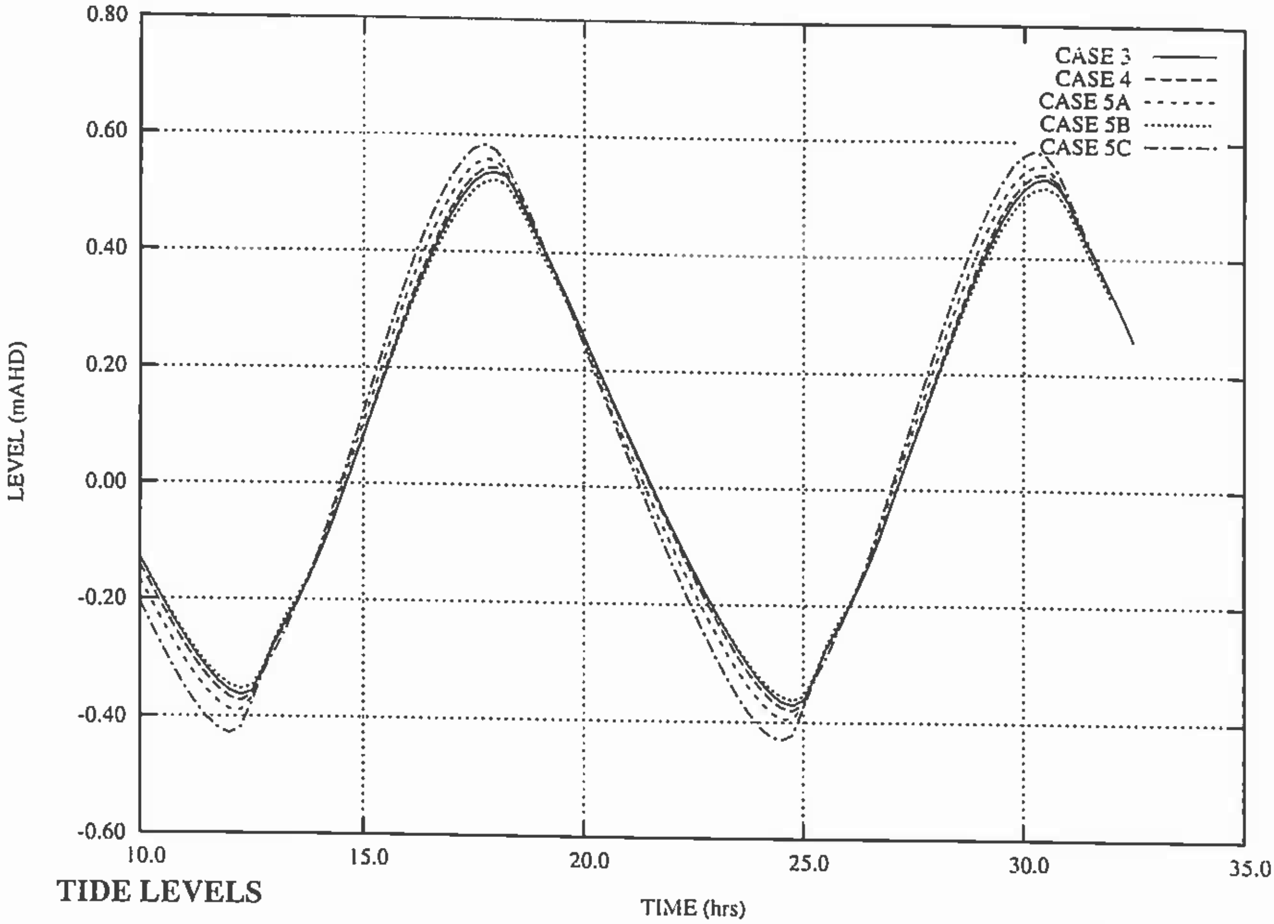


TIDAL FLOWS

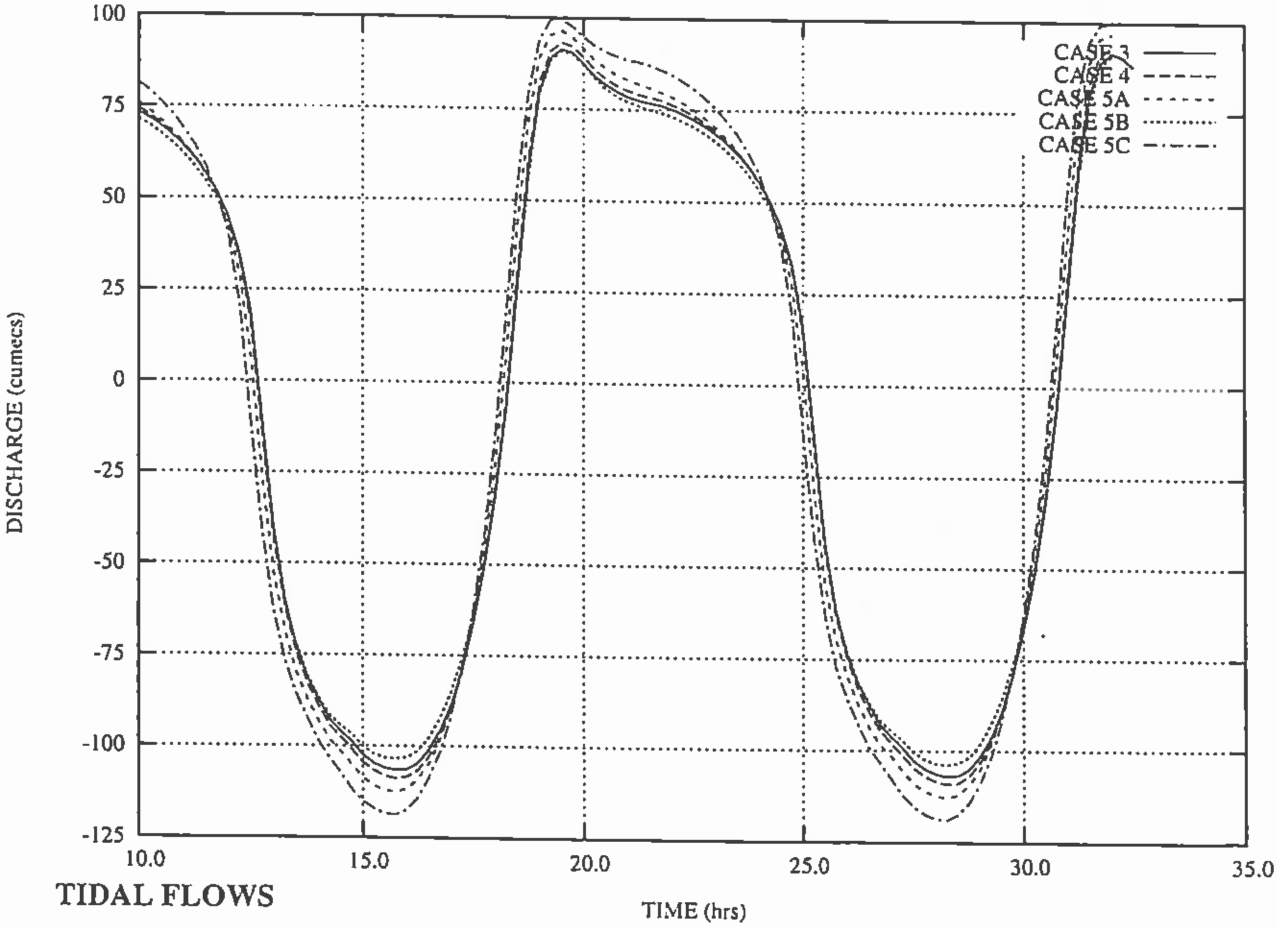
DREDGING AND BYPASSING IMPACTS
LEVELS AND FLOWS - BARNEYS POINT

Figure E.14

WBM



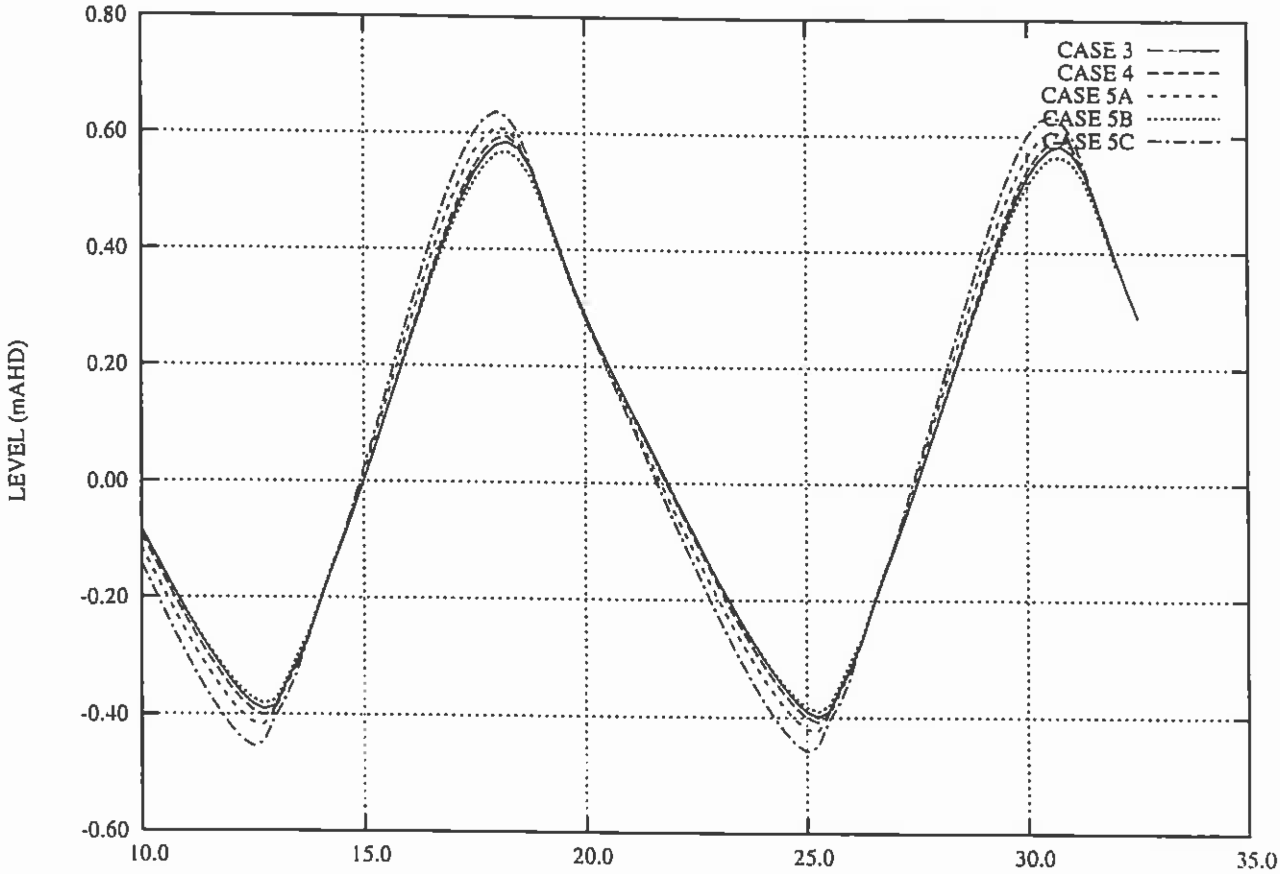
TIDE LEVELS



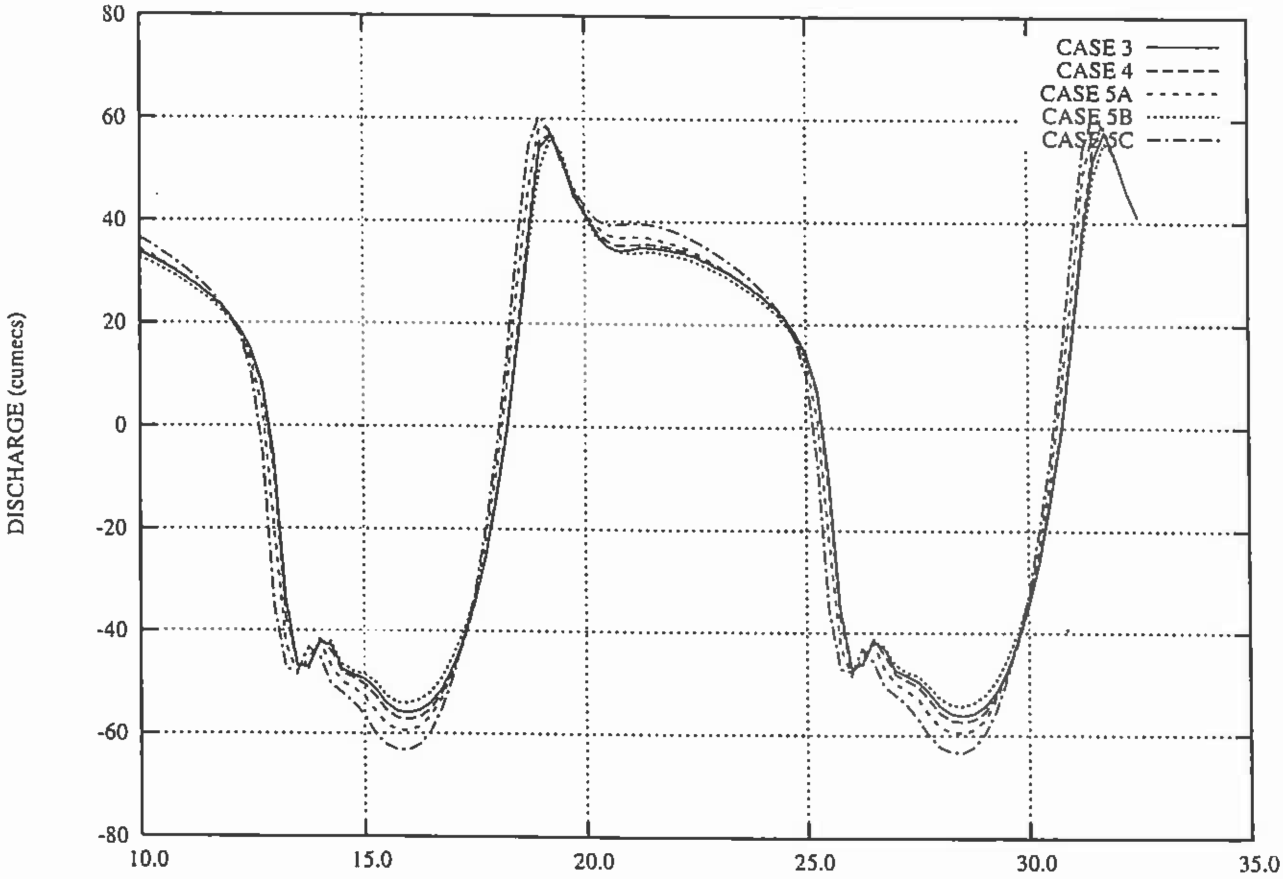
TIDAL FLOWS

DREDGING AND BYPASSING IMPACTS
LEVELS AND FLOWS - TUMBULGUM

Figure E.15



TIDE LEVELS

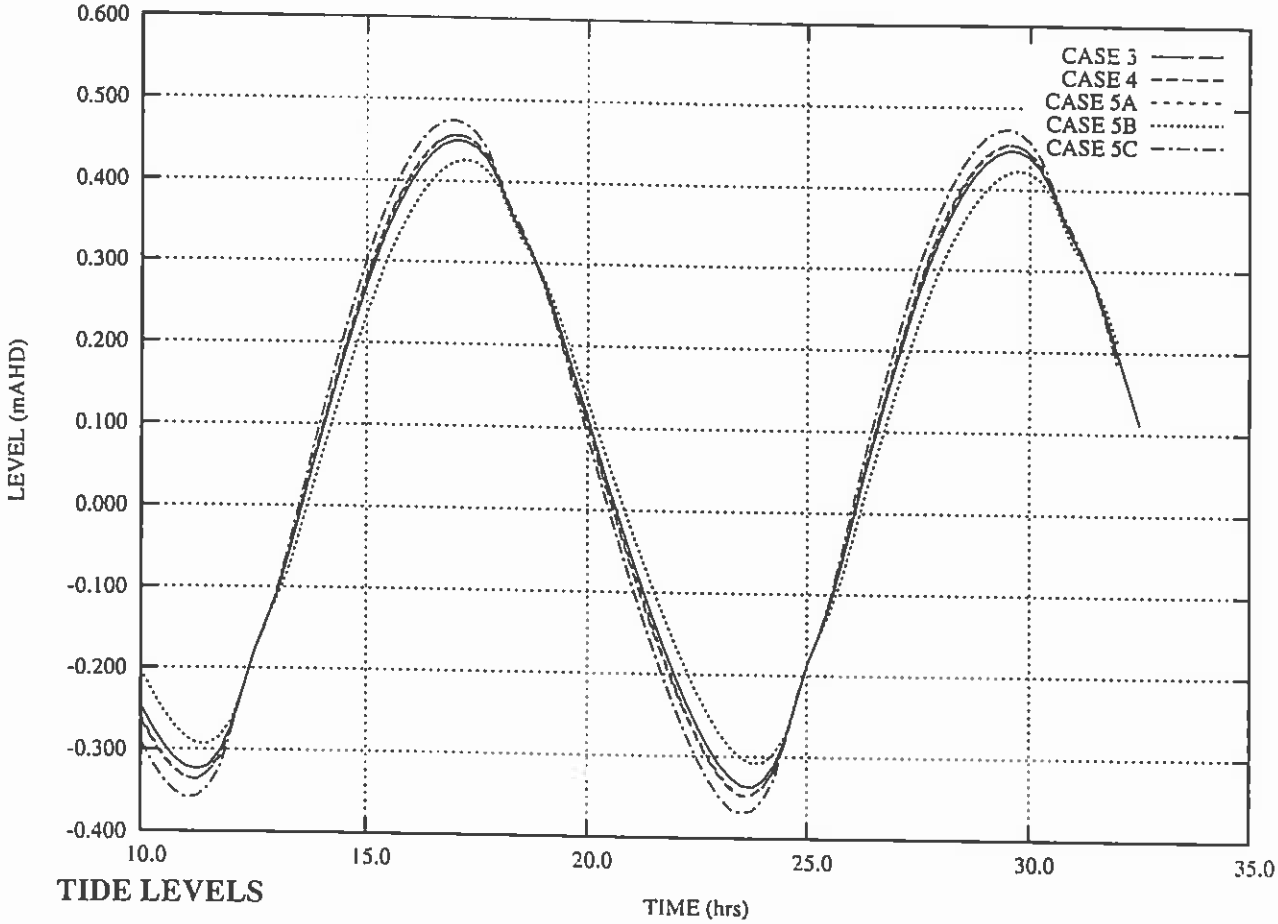


TIDAL FLOWS

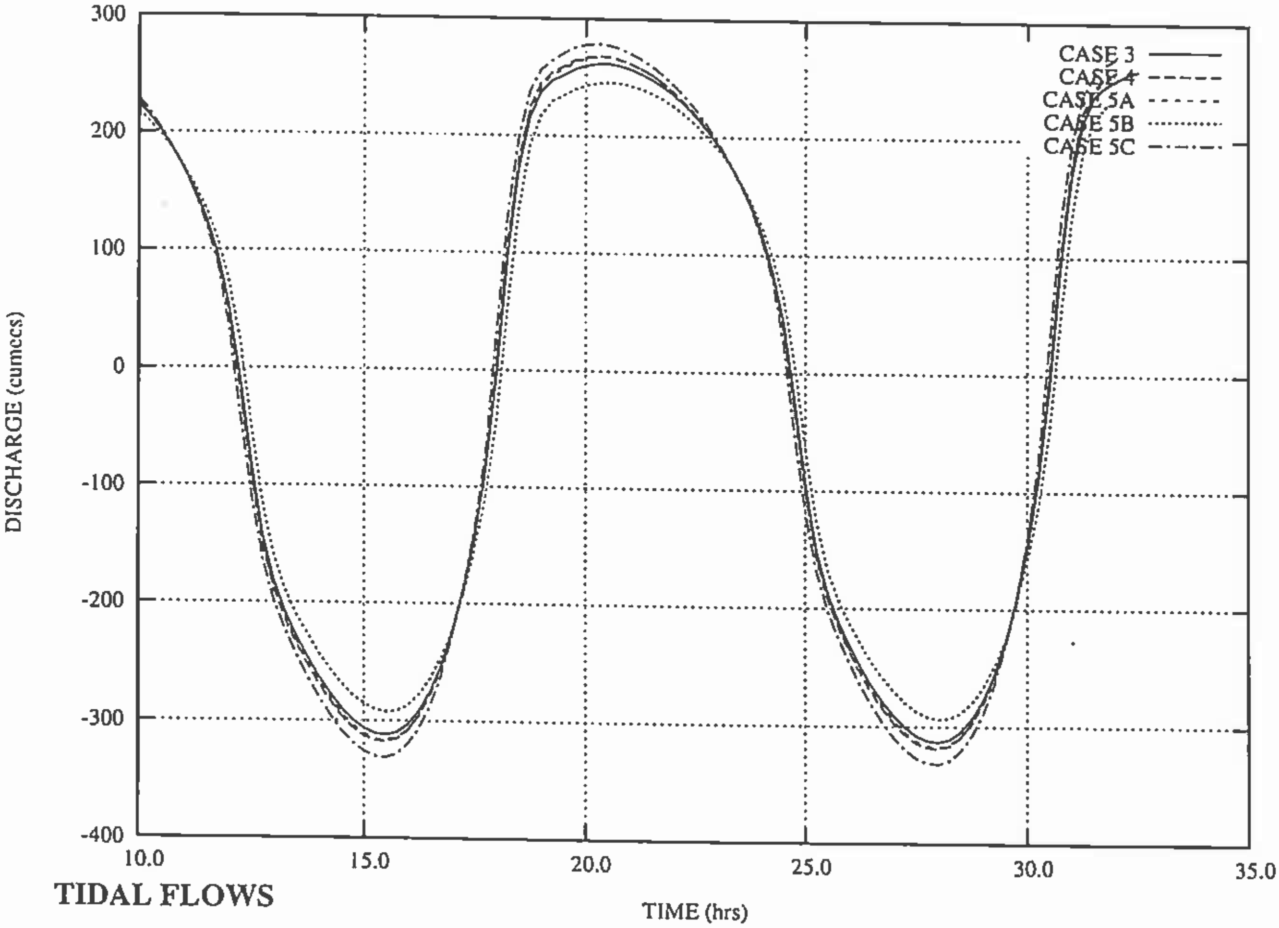
DREDGING AND BYPASSING IMPACTS
LEVELS AND FLOWS - MURWILLUMBAH

Figure
E.16

WBM



TIDE LEVELS



TIDAL FLOWS

DREDGING AND BYPASSING IMPACTS
LEVELS AND FLOWS - DRYDOCK

Figure E.18