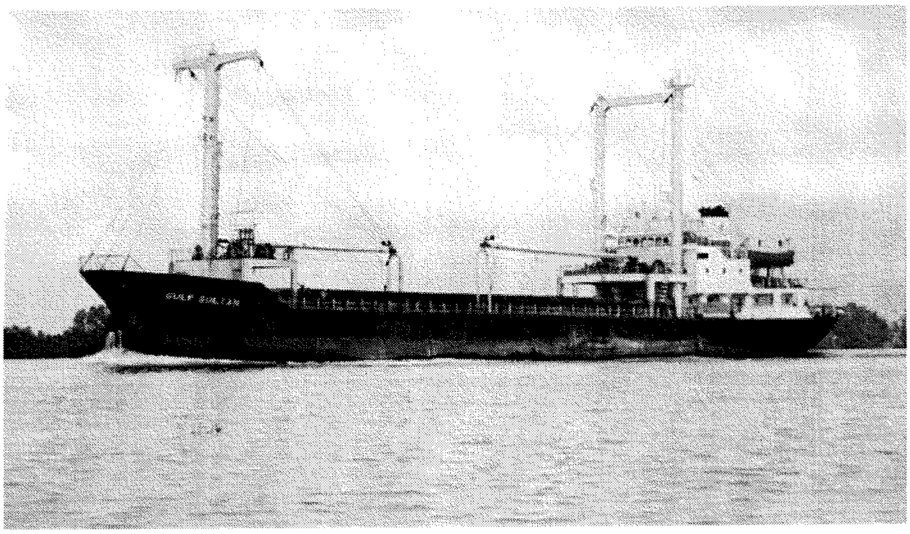
Accessibility of the port of Palembang

Riverworks and dredging in the Musi river

August 29, 1997

S.A. Heukelom



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Preface

As part of the cooperation between the DUT (Delft University of Technology) and ITS (Institut Technogi sephulu nopember Surabaya) research has been done on the Musi river, by order of the harbor authorities of Sumatra (Pt. Pelabuhan II). The larger ships that call on the Port of Palembang are having difficulties sailing up the Musi river because of their large draught. Therefore the possibilities of an increase of the navigational depth of the Musi river is investigated by ITS and DUT.

As part of this cooperation two studies have been done. A *VTS* (*Vessel Traffic Study*) by R. Brans and a *River and Dredging Study* by the author.

Part of the preparation for this thesis was a two month stay at the ITS in Surabaya, Java, Indonesia where we joined the Faculty of Civil Engineering for analyzing the project in cooperation with the project group of the ITS. We also collected data for our thesis and went to the Musi river at Sumatra to investigate the situation on site and to make appointments with the different terminals to gather additional information on the subject.

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Rectification

The output of Musi River Model (chapter 3) seemed to be sufficiently accurate for this study concerning the range and phase of the water levels and the range of the water velocities on which the sediment transport calculations (chapter 4) and the dredging cost calculation (chapter 6) were based. When the discharge was checked it has find out that, the net downstream discharge during one tidal cycle near Palembang was too small.

The implementation of the sections (23, 24, 25, 33, 32, 31, 30 and 40 to 48 see figure 3-6 ~~Error! Reference source not found.~~) upstream of Palembang was improved, which resulted in a correct net discharge (Q = 300 m^3/s) during one tidal cycle downstream of Palembang. Due to the late implementation of the mentioned improvements it was only possible to implement the corrections in the Musi River Model (chapter 3).

As a result of the changed output of the model the mentioned sediment transport calculations and the dredging cost calculation will change, although the final cost calculation (chapter 7) will hardly change and the final conclusions (chapter 8) will not change. This is based on the following facts:

- the calculations of the amount of soils to be dredged for increased depths are based on the dredging history and they remain the same.
- 15 % of the total calculation that will change (sediment transport and dredging costs) were based on the DUFLOW results.
- the dredging costs amounts 7 % of the total costs for the optimal river depth and therefor the optimal navigational depth is largely determined by the shipping costs.

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A(x,H)	cross-sectional flow area [m^2]
b(x,H)	cross-sectional flow width [m^2]
B(x,H)	cross-sectional storage width [m^2]
B	bulking factor [-]
C(x,H)	coefficient of Chézy [$m^{½}/s$]
d	water depth [m]
D_{50}	grain size [mm]
f_b	mechanical breakdown factor [-]
f_c	cycle factor [-]
f_d	delay factor [-]
f_e	proportion of hopper filled [-]
f_o	operational factor [-]
f_t	traffic delay factor [-]
f_w	weather delay factor [-]
g_d	distance to disposal ground [-]
g	acceleration due to gravity [m/s^2]
H_c	hopper capacity [m^3]
H (x,t)	water level with respect to the reference level [m]
i	bottom slope [-]
l	length of dredging area [km]
L	cross-sectional length [m]
P	output [m^3/hr]
P_{max}	maximum potential output [m^3/hr]
Q(x,t)	discharge at location x and time t [m^3/s]

$R(x,H)$ hydraulic radius of cross-section [m]

S_i transport capacity [m^3/s]

t time [s]

t_d time for soil disposal [hr]

t_l time to load hoppers [hr]

t_t time taken to turn dredger [hr]

U_m modified productive unit [m^3]

$v(x,t)$ mean velocity (averaged over the cross-sectional area) [m/s]

V_g fully laden sailing speed [km/hr]

V_i (yearly) transported sediment volume [m^3/year]

w fall velocity [m/s]

$w(t)$ windl velocity [m/s]

x distance as measured along the channel axis [m]

$\Phi(t)$ wind direction [°]

$\phi(x)$ direction of channel axis, measured clockwise from the north [°]

$\gamma(x)$ wind conversion coefficient [-]

α correction factor for non-uniformity of the velocity distribution in the advection term [-]

Δ relative density (=1,65)

ν kinematic viscosity (1,0e-6)

1 Horse Power = 736 Joule/ sec = 0.736 [kW]

1. Introduction

1.1 General description

The Indonesian Archipelago consists of about 13,000 islands with a total coastline of more than 81,000 km. Two-thirds of the territory of the Republic of Indonesia is covered by seas or oceans (see Figure 1—1). Its location is in the humid tropical zone with a monsoon-type climate. As a result of the growth of the Indonesian population and economy, there has been an increase in the cargo flows within the country. As a transport node Palembang, a large port, serves several functions, i.e. a transport, commercial and industrial function. The development of the Port of Palembang is hindered , among other things, by the insufficient depth of the Musi river, used by large ocean-going vessels to enter the Port of Palembang.

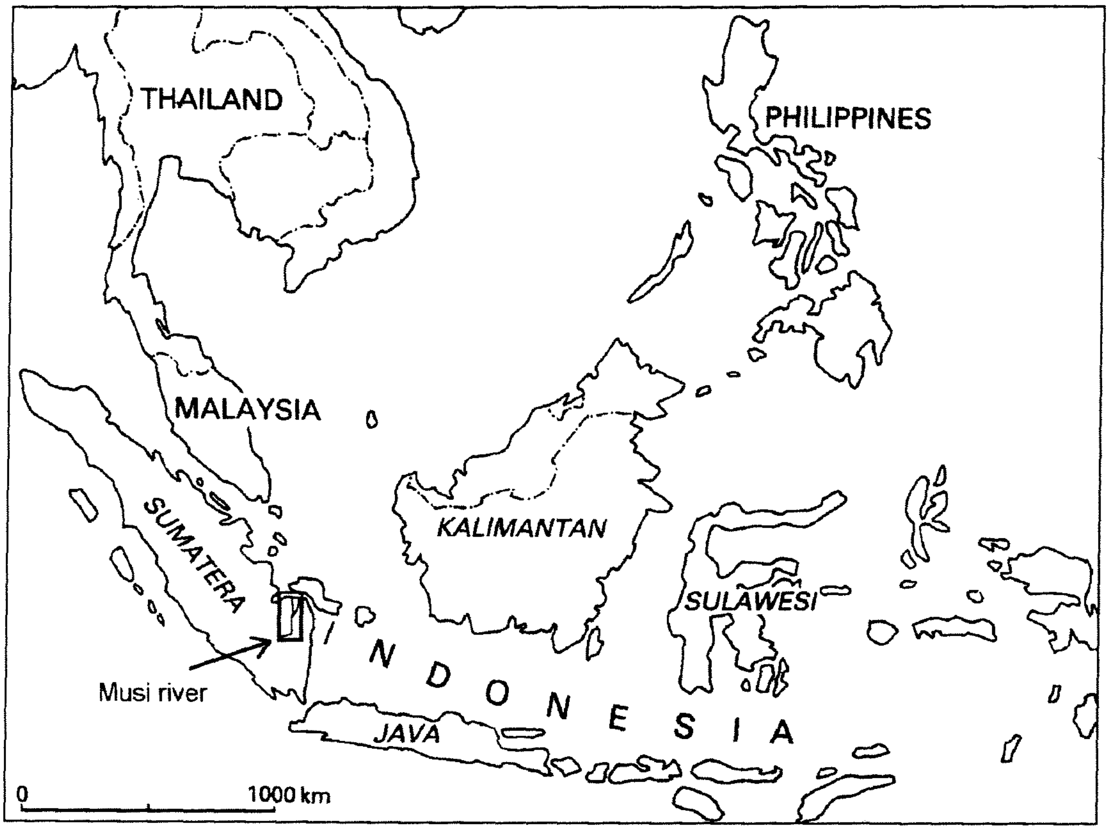

Figure 1—1 The Indonesian Archipelago, part of Southeast Asia. The island of Sumatra is located in Eastern Indonesia.

In this report the possibilities of an increase of the navigational depth of the Musi river will be investigated. Several combinations of riverworks and maintenance levels will be compared using the computer program DUFLOW. The aspect of costs plays an important role. Effects of dredging and riverworks on the Musi river between Palembang and the sea will be examined.

1.1.1 Musi river

The Musi river is situated in the south of Sumatra, one of the five major Indonesian islands. A large part of Sumatra is still covered with rain forest. The eastern third of the island consists of lowlands, giving way to vast areas of swampland and estuarine mangrove forest. This region has a tropical climate with a dry and a wet season. The Musi river rises in the western mountains of South Sumatra and has a catchment basin of 4×10^4 km^2, with a rainfall of around 3200 mm per year.

The sea immediately surroundings of the estuary is generally quite calm. Only a combination of an extreme (northern) wind during the west-monsoon in the rainy season and the river's ebb current (from the opposite direction) can cause a rough sea with short waves (choppy sea) during a few hours. An important factor which influences the currents and the water levels of the Musi[1] river is the tide. Especially the diurnal tide has a strong influence, i.e. even in Palembang the flow of the river reverses due to the diurnal tide (see Figure 2—3).

[1] "Musi" means tide in the Indonesian language.

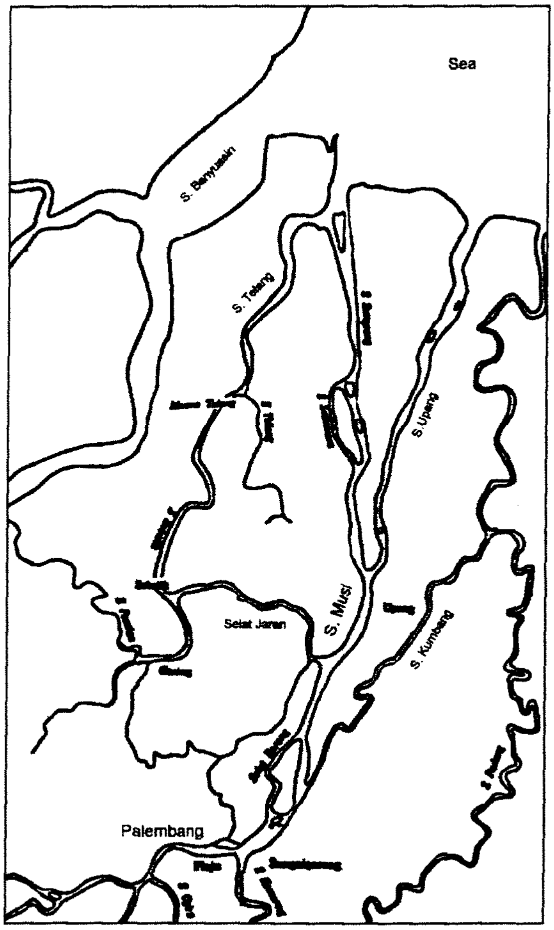

Figure 1—2 The Musi river and its tributaries.

1.1.2 Palembang

Palembang is located approximately 100 km inland and is connected to the sea by the Musi river. The Port of Palembang began as a trade centre established by the Dutch V.O.C. in the 16 th. century. This trade centre has now become the largest and most important port on the east coast of Sumatra.

Nowadays, Palembang is an industrial city with a population of 1.5 million and it is the second largest city of Sumatra. Palembang plays an important role in the economic development of Central Indonesia. For this there are two main reasons. First of all, Sumatra is rich in mineral resources, which are exported through the Port of Palembang. Secondly, the island benefits from the nearby island of Java, a densely populated island with a vast economic growth.

Besides the export of rubber, coffee, pepper and pineapples the main activities are the transfer of crude oil, the production and the transshipment of fertilizer, the transsshipment of coal and cement manufacture. Because of these industries the shipping traffic is heavy on the Musi river, ranging from smaller ships for local trade to large vessels (up to 160 m, with a draught of 6.0 m, see Table 6-3) for the oil industry.

Due to the growth of the Indonesian population and economy, there has been an increase in the cargo flows within the country. For the Port of Palembang with its regional function, this means that the number of vessels calling the port increases every year and so does the throughput of the different terminals. The terminals in the Port of Palembang require an increase of the depth of the Musi river navigation channel, in order to prevent congestion and make more efficient use of the fleet capacity. The larger ships are loaded for 50 % at the most, because of their too large draught when fully loaded. An increased depth should lead to an increase in the feasible average draught (more than 50 %) of the vessels and therefore to lower costs.

For this, the port and navigation channel management, the Indonesian port authority Pt. Pelabuhan, want to investigate the possibilities of a depth increase of the Musi river navigation channel, from the sea to the Port of Palembang.

1.2 Problem analysis

Ships sailing to and from the port of Palembang follow the Musi river. A number of shoals in the river and a bar at the river mouth limit the navigability of the Musi river. At low tide the large ships have too large a draught, which means they have to wait at the river mouth for high tide before they can sail up the Musi river.

While the ships are waiting at the outerbar (or in the port of Palembang, to sail to the sea) they are non-productive. The terminal for which the ship is heading has to pay part of the costs of the lost hours to the shipping company. These costs are the so-called 'waiting costs'.

An increase of the tidal window will reduce the waiting time and consequently reduce the waiting costs for these ships. This can be achieved by increasing the navigational depth of the Musi river by means of river improvements and dredging.

As result of a bigger tidal window the maximum draught could also be increased, since at present the large vessels can not use their maximum loading capacity.

However, that is a matter of economic optimization of the usage of the relevant vessels, which is the subject of the 'Vessel Traffic Study', done by R. Brans [3].

There are three possible ways to increase the river's depth. The first option, dredging, is a less permanent way to increase the river's depth, since maintenance dredging will have to be performed every year. The second option, applying riverworks achieves a more permanent increase in depth, but without dredging it might take more than 50 years before the required depth is achieved. A combination of both is the third option.

The total project costs include waiting time costs, riverwork costs and dredging costs. In this study 'Riverworks and Dredging on the Musi river', research will be done on the effects of both riverworks and dredging as regards the increase of the river's depth. The following cases will be reviewed:

- No riverworks — maintaining depth through dredging only
- Set 1 of riverworks — groynes at shoals along the Musi river to constrict the flow width and maintaining the rivers depth through dredging
- Set 2 of riverworks — groynes at shoals along the river + closure of one branch around P. Payung (island in the river mouth) and maintaining the rivers depth through dredging

The above mentioned cases will be analyzed for different bottom levels, where the actual maintenance level is LWS -6.5 m (see Figure 2—4), in the following way:

	Depth below LWS	Riverworks	Capital dredging	Maintenance dredging
Alternative 0	6.5	no	..	..
No riverworks	7.0	no	..	..
	7.5	no	..	..
Alternative 1	6.5	..	..	..
Set 1 of	7.0	..	..	..
riverworks	7.5	..	..	..
Alternative 2	6.5	..	..	..
Set 2 of	7.0	..	..	..
riverworks	7.5	..	..	..

Figure 1—3 Costs of three possible alternatives.

On the basis of the resulting graphs an optimal combination of riverworks, dredging and the involved costs will be determined. In combination with the results of the 'Vessel Traffic Study' by R. Brans, i.e. analysis of the waiting costs, this will give an optimum maintenance depth for the Musi river.

1.3 Objectives of the thesis

The objective of this study is to find a solution for the problem described in the previous paragraph, i.e. what is the most suitable way, with regard to the costs and feasibility, to improve the navigational depth of the Musi river.

Research will be done on the following subjects:

- determination of alternative riverworks, including construction method and resulting costs.
- construction of a simulation model of the Musi river with the program DUFLOW, to simulate changes in the river as a result of the riverworks and dredging.

- calculation of the sediment transport in the different cases.
- determination of (to be) used dredging equipment and dumping location of spoil.
- calculation of the amount of material to be dredged (capital and maintenance) and the resulting costs.
- determination of the lowest level of costs of riverworks and dredging.

These main subjects play a role in this study. To provide some more insight into how they are related and how they will lead to the final solution they have been schematized in Figure 1—4.

1.4 Contents of the thesis.

This thesis is divided into 7 chapters. Each chapter deals with one of the main subjects mentioned in the previous paragraph.

Chapter 1 gives an introduction on the Musi river and the problems encountered in the entrance channel of the Port of Palembang. The present-day situation of the Musi river, as well as the available data, will be described in Chapter 2. A simulation model of the Musi river with the DUFLOW program is discussed in Chapter 3. In Chapter 4 a sediment transport calculation will be made to predict the amount of sediment that will have to be dredged in the future. In Chapter 5 the possible application of riverworks, such as groynes, will be discussed. Chapter 6 will go into the aspects of dredging on the Musi river, i.e. methods, dredged amounts and costs. Final conclusions and recommendations are given in Chapter 8.

Chapter 9 will contain the synthesis of this thesis.

The various subjects treated in this thesis are dealt with in the chapters mentioned above. The relations between the different subjects are set out in the following diagram:

Relation diagram

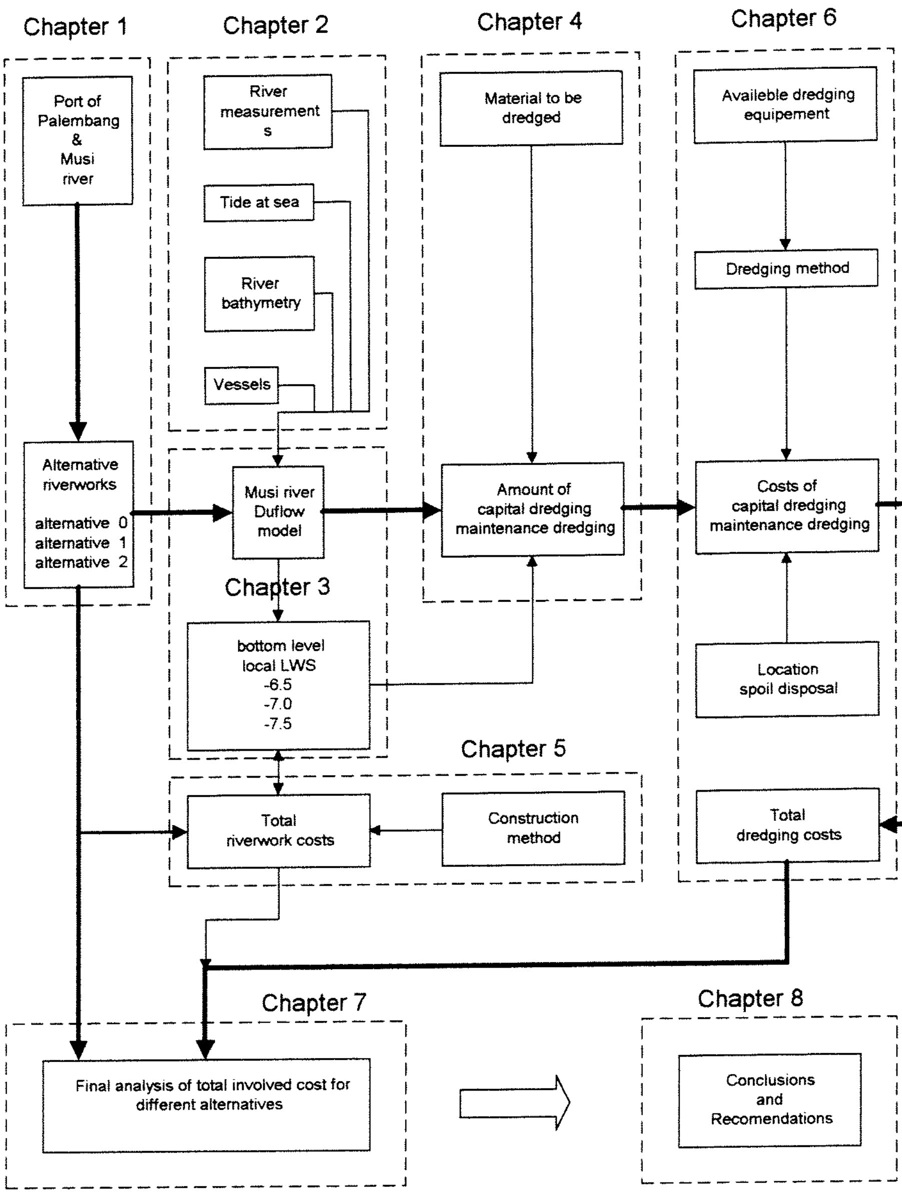

Figure 1—4 Relation scheme of involved subjects.

2. Musi river

2.1 Available data and literature

The available data that have been considered for the river analyses comprise:

- seasons [9],
- the tide tables [6],
- the sounding map [7] and
- the results of two measurement campaigns (see paragraph 2.2) .

A previous feasibility study on the Port of Palembang done by Haskoning in1984 [10] has also been used.

Season

As has been stated before, the Musi river is influenced by the upstream fresh water discharge and the downstream sea state.

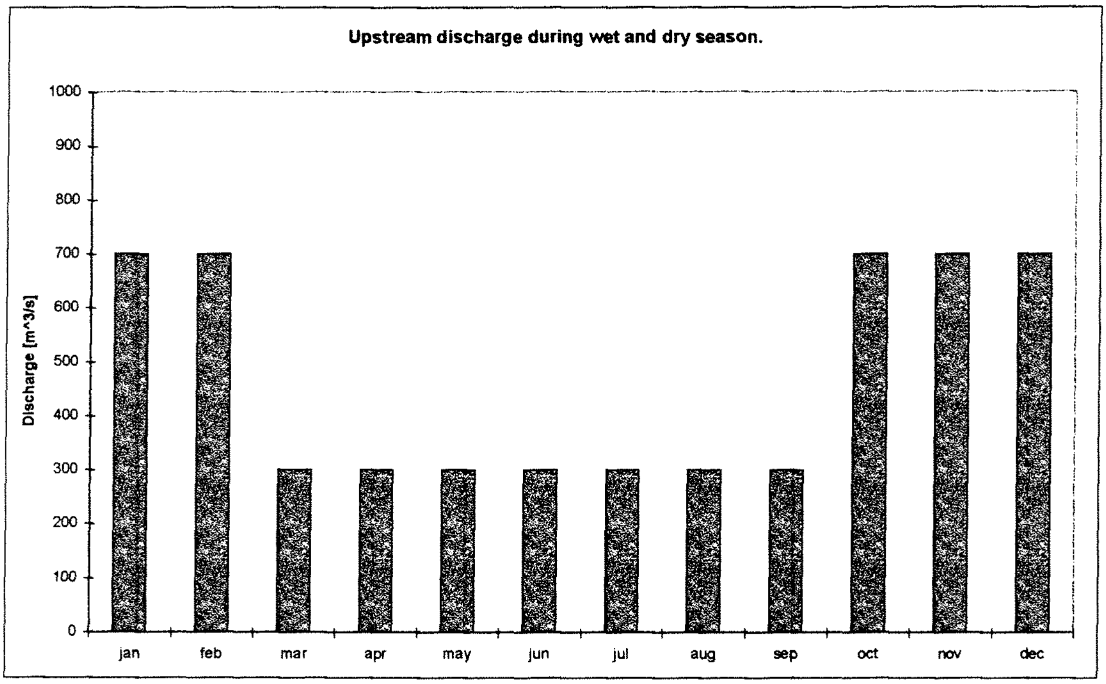

Figure 2—1 Schematization of upstream discharge

The upstream discharge throughout the year can be divided roughly into 2 situations:

- a wet season from October until February
- a dry season from March until September (see Figure 2—1)

Tide

The tidal period at sea is approximately 24 hours (see Figure 2—3) and one complete spring-neap cycle takes 14 days. The tidal amplitude is different for spring tide and neap tide. During spring tide, when the lowest water levels occur and ships therefore suffer the most hindrance, the tidal amplitude is approximately 1.25 m (see Figure 2—2). Water levels are referred to Chart Datum, which is 19 dm below MSL.

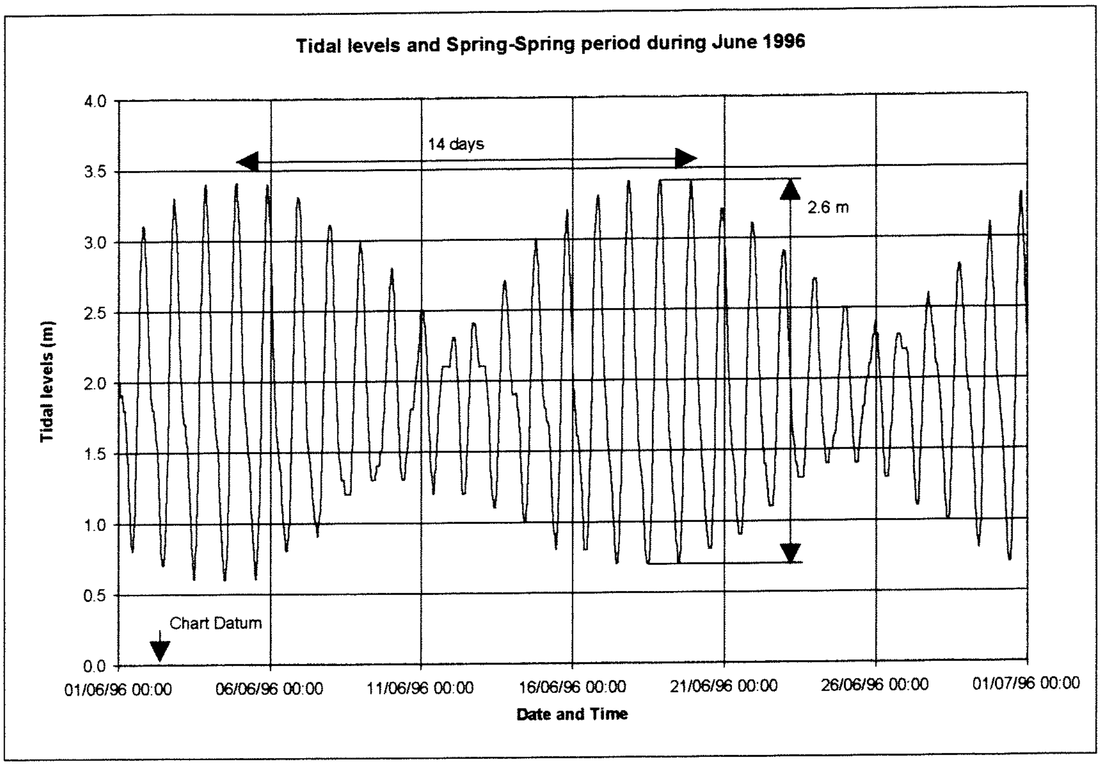

Figure 2—2 The tidal variation at sea near the mouth of the Musi river, during June 1996.

There is still a significant tidal range in Palembang as can be seen in see Figure 2—3. In Palembang still 90 % of the tidal range of the river mouth is present. The levels of 'Sea' are from the tide tables and those of 'Dhermaga BBC' (location 1, Figure 2—5) are measured levels in Palembang.

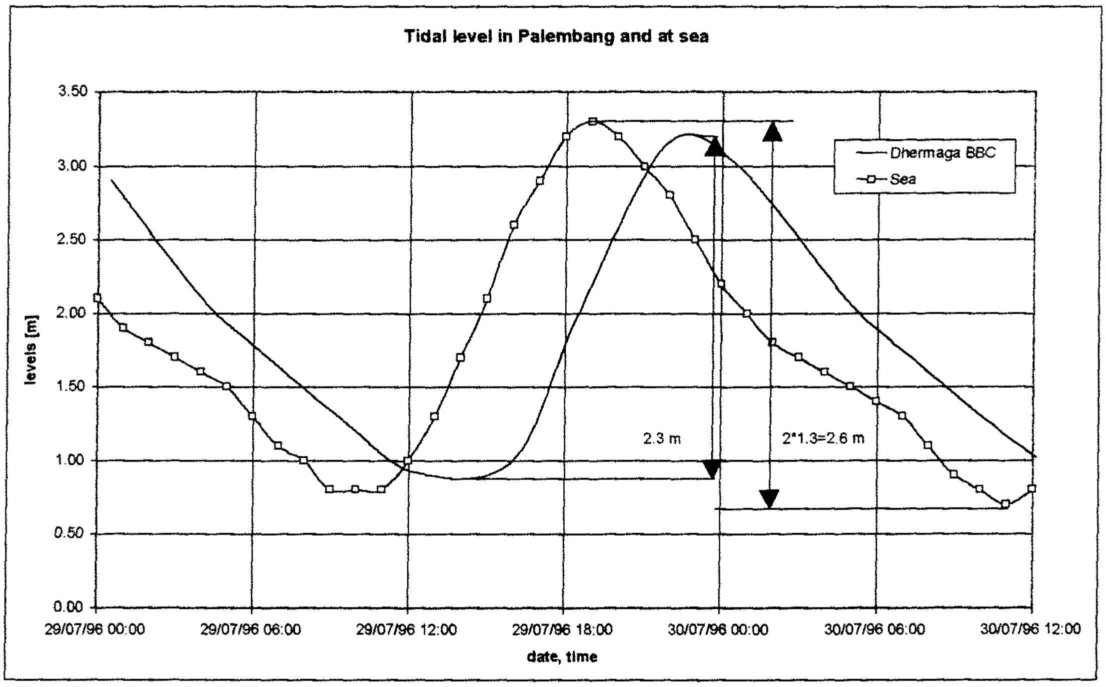

Figure 2—3 The tidal phase and amplitude at sea and near Palembang[1] during the spring tide period in July.

Bathymetry

The longitudinal profile (see Figure 2—4) of the Musi river from Palembang to the sea is composed on basis of the navigation map [7]. For each kilometre from the Boom Baru terminal at Palembang to the mouth of the Musi river the depths have been taken, which are referred to local LWS.

[1] The Dhermaga BBC curve has not been referred to LWS in the mouth of the Musi river. It is composed of 'raw' measurement data. Therefore only the tidal phase and amplitude of this curve are relevant and not the levels.

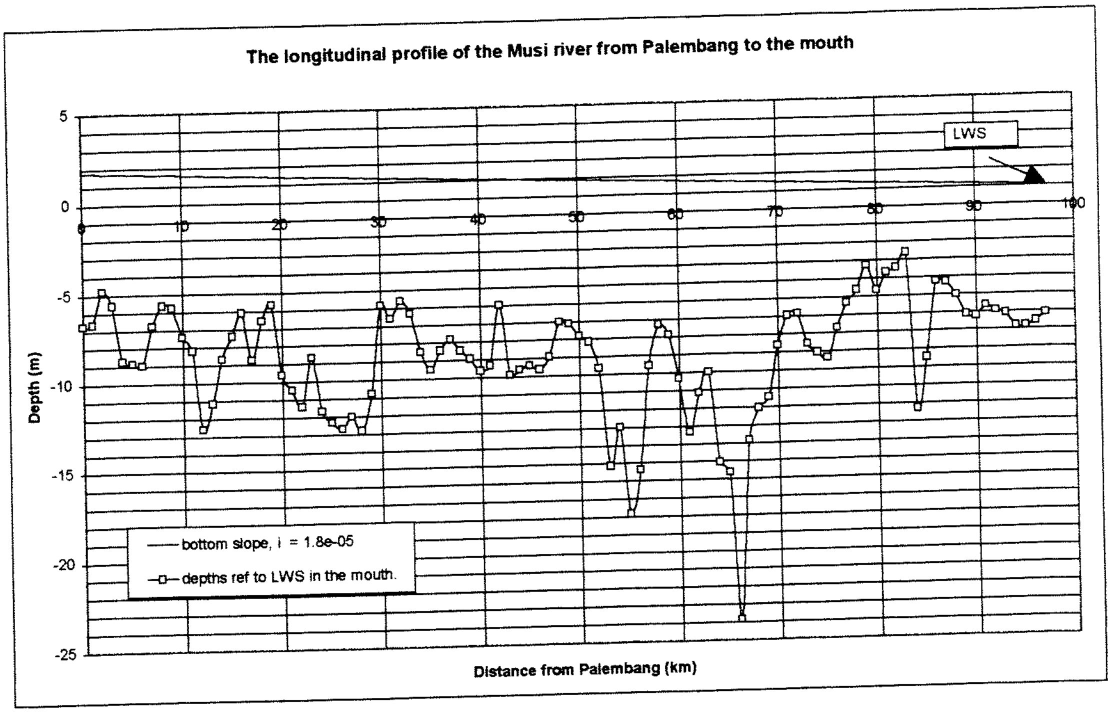

Figure 2—4 Longitudinal profile of the Musi river.

The DUFLOW model (3) requires a single reference level along the whole river. This means the locally referred bottom levels have to be corrected. The Musi river has an average bottom slope of $i_b = 1.8*10^{-5}$ [10]. Adding up the bottom elevation as a result of the slope and the locally referred depths along the Musi river, yields bottom depths referred to one reference level for the whole river, i.e. LWS in the mouth.

2.2 Measurements

There have been 2 measurement campaigns, one in January, dry season, and one in July 1996, wet season, (see Table 2-1). The campaigns covered 90 km along the Musi river, from Dermaga BBC in Palembang to the outerbar, which means the outerbar itself was not included. Both campaigns included a spring and a neap tide period.

	Neap tide	Spring tide
January (wet season)	14/15	22/23
July (dry season)	23/25	29/31

Table 2-1 Dates on which the measurements were carried out.

The paragraphs 2.2.2 and 2.2.3 deals with the measurement methods, for the water levels and current velocities and the consequences of these methods on the use of their results for calibration of the DUFLOW model of the Musi river.

2.2.1 Measurement locations

During the two measurement campaigns, carried out in connection with this study, a total of 10 locations were surveyed (see Figure 2—5):

1. Dermaga BBC (*)
2. Sungai Lais (*)
3. Ayer Kumbang
4. Selat Borang
5. Selat Jaran (*)
6. Sungai Upang
7. Pulau Ayam (*)
8. Parit 12
9. Pulau Payung Kanan (right)
10. Pulau Payung Kiri (left) (*)

Of these locations, 5 were surveyed in January[1] 1996 and all 10 were surveyed in July[2] 1996. At these locations the following data was collected:

1. water levels
2. current velocities
3. sediment samples
4. salinity concentrations

This chapter deals with the measurement methods and the consequences of these methods on the use of their results for calibration of the DUFLOW model for the Musi river. In the following figure the measurement locations are numbered and marked with a rectangle.

[1] The locations marked with (*) were surveyed during both January and July 1996.

[2] It should be noted that the bathymetry measurements were only carried out once, i.e. during the wet season (January) or during the dry season (July).

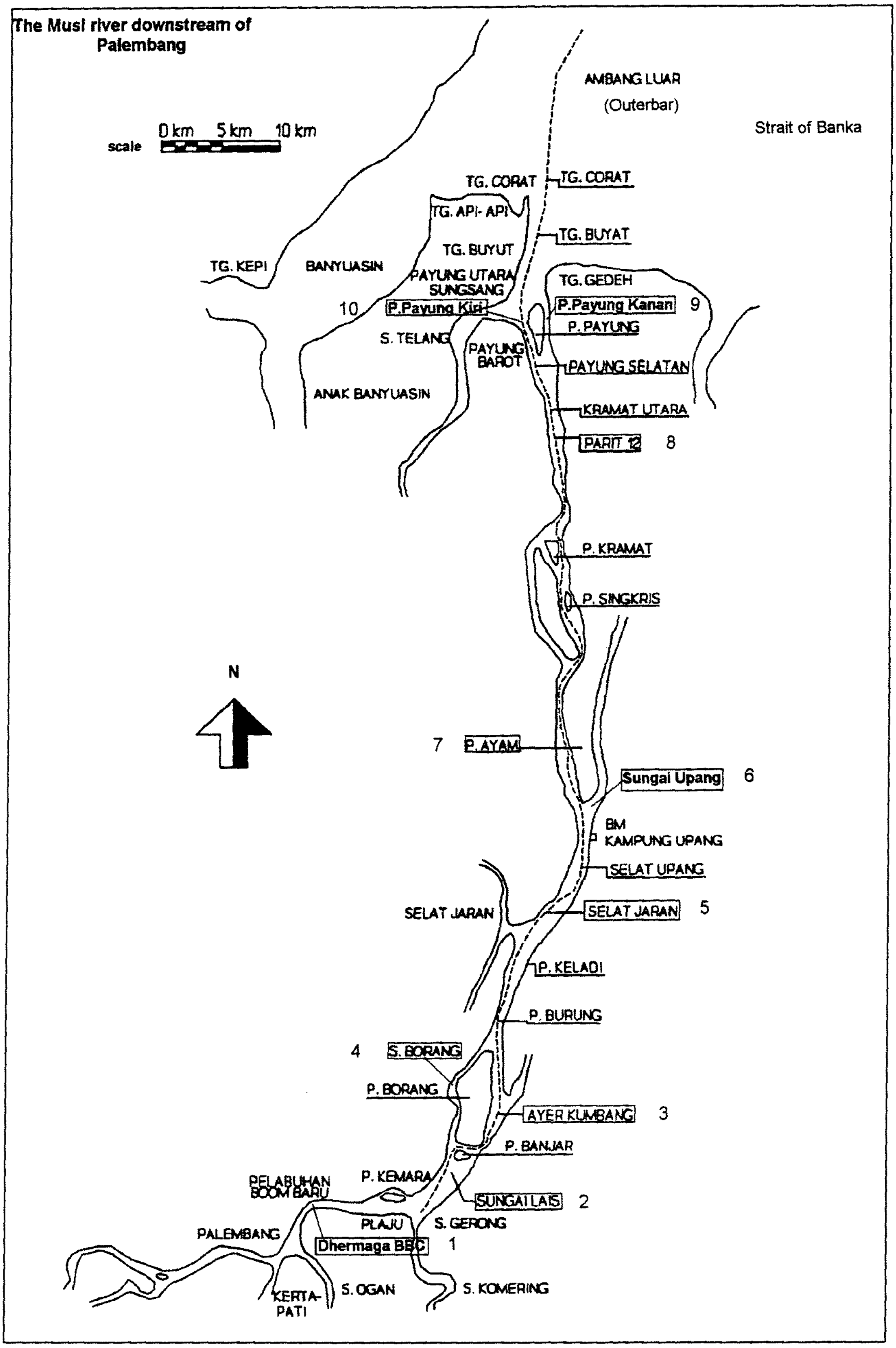

Figure 2—5 Measurement locations along the Musi river downstream of Palembang.

2.2.2 Water levels

The water level measurements were carried out according to the following principle. First at each location a local reference level was determined, the so called 'peilschaal'[1], which is a local reference level.

Then the measurements were taken and referred to this local 'peilschaal' (see Figure 2—6). The measurements should be adjusted to account for the average bottom slope ($i = 1.8*10^{-5}$ according to [10]) to obtain the water levels referred to LWS in the mouth of the Musi river (see Figure 2—7).

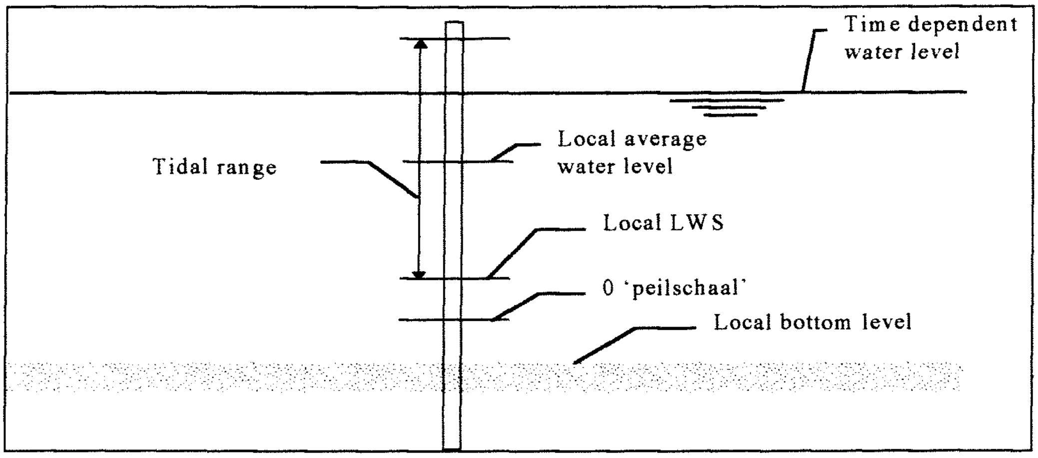

Figure 2—6 Illustration of water level measurement method.

The use of this method has certain consequences when the measurement results are used to calibrate the DUFLOW model of the Musi river. In the DUFLOW model the reference level equals LWS at sea near the mouth of the Musi river. Therefore more exact information about the differences in the local mean river levels in each location, compared to LWS in the mouth of the Musi river, is needed (see Figure 2—7). Unfortunately this information has not yet been obtained[2]. As a consequence, only the *tidal range* and the *tidal phase* of the water level measurements can be used to calibrate the DUFLOW model of the Musi river.

[1] In the report of resulting data of the measurement campaigns 'peilschaal' was introduced.

[2] March, 1997.

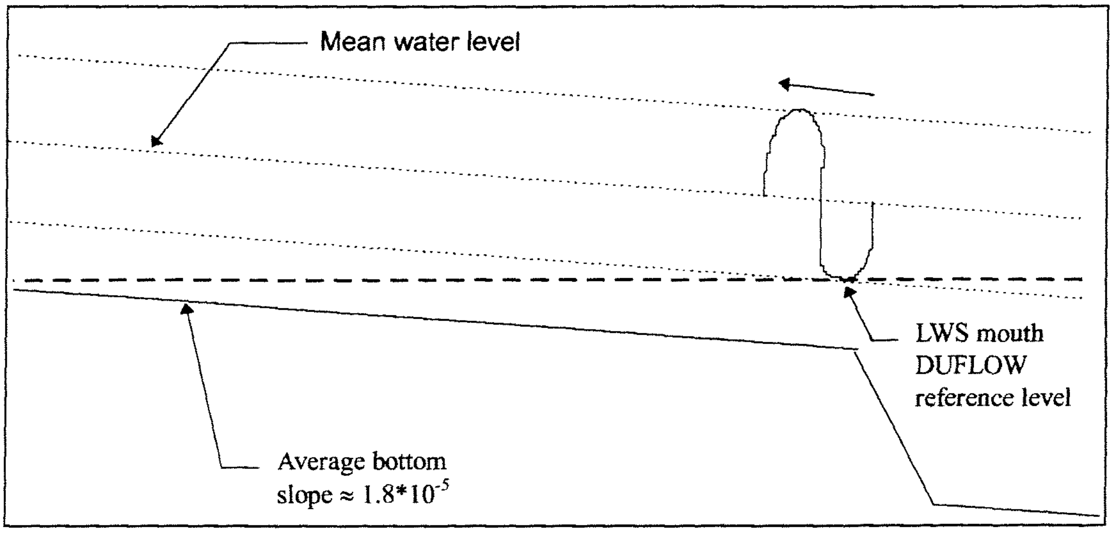

Figure 2—7 Correction of the local water level measurements to account for the average bottom slope.

2.2.3 Current velocities and discharges

The velocity measurements were carried out in a different way in January and July. First the general method to measure the velocities will be described, after which the differences between the January and July methods will be explained.

The general method used has been the so called 3-point Czechoslovakia-method (see Figure 2—8) [19]. Although this method is designed to measure velocities and calculate discharges in non-tidal rivers, it is used because in practice it gives good results.

The first step in this method is to measure the velocities at 20%, 60% and 80% of the depth on the left-hand side, in the middle and on the right-hand side of the cross section[1]. This is done with directional velocity meters. These meters not only measure the value of the velocity, but also the direction of the current. It should be noted, however, that there is no accurate information about the direction of the cross-sections available.

[1] It is not known to the author whether average depth or actual depth is used.

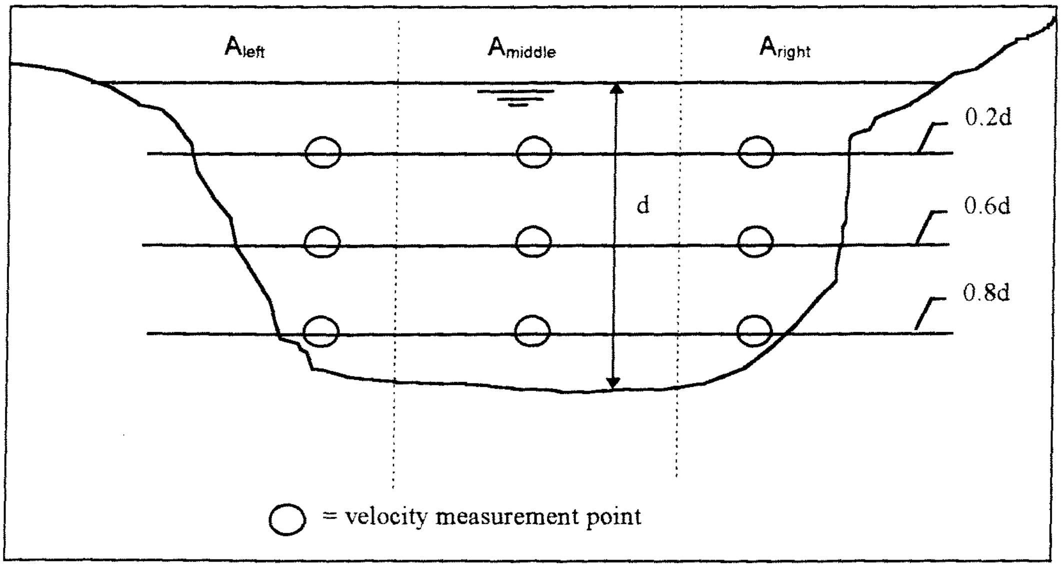

Figure 2—8 The 3-point Czechoslovakia-method to measure velocities and calculate discharges.

The next step is to calculate the cross section areas of the left-hand side, the middle-hand side and of the right-hand side of the cross section. This is done graphically.

Then the velocity on the left-hand side, in the middle-hand side and on the right-hand side of the cross section can be calculated according to the following formula:

$$\bar{v} = \frac{1}{4}(v_{0.2} + 2v_{0.6} + v_{0.8})$$

The last step in calculating the discharges consists of the multiplication of the velocities and the cross section areas and adding them up according to the following formula:

$$Q_{cross-section} = \sum_{n=1}^{n=3} \bar{v}_n A_n$$

in which:

1 = left-hand section

2 = middle section

3 = right-hand section

In *January,* the measurements were not carried out in all 3 sections, but in only 2 at a time. They were always taken in the middle section and on one of either side sections. Moreover the velocity measurements in January only covered 24 hours and were taken at an average interval of 3 hours (8 incomplete series). Because of these limitations, the results cannot be used for the calibration of the DUFLOW model.

In *July,* the measurements were carried out in all 3 sections (left, middle and right) and also covered a longer period (3 days during spring tide). These results are therefore more useful for calibration of the DUFLOW model of the Musi river than the January measurements. However, some reservations have to be made:

- The time interval between measurements is not always constant.
- Some velocity measurement values are lacking.
- The direction of the cross sections is not exactly known.

2.2.4 Soil

In general the soil type along the Musi river can be classified according to two classes, i.e. sand (fine to medium) and silts in combination with soft to very soft clays [see Appendix A]. Fine to medium sand soil can generally be found within 50 km downstream of the Port of Palembang (Dermaga BBC berth) to Pulau Ayam. In this there are two places where the soil are generally silt to very soft clay, i.e. between Sungai Lais and Ayer Kumbang and Selat Jawa (35 km. from downstream of Dermaga BBC). Further downstream the soil of the Musi river are dominantly silt, i.e. 40 km up to Pulau Payung, including a few locations which contain sand (see Figure 2—9).

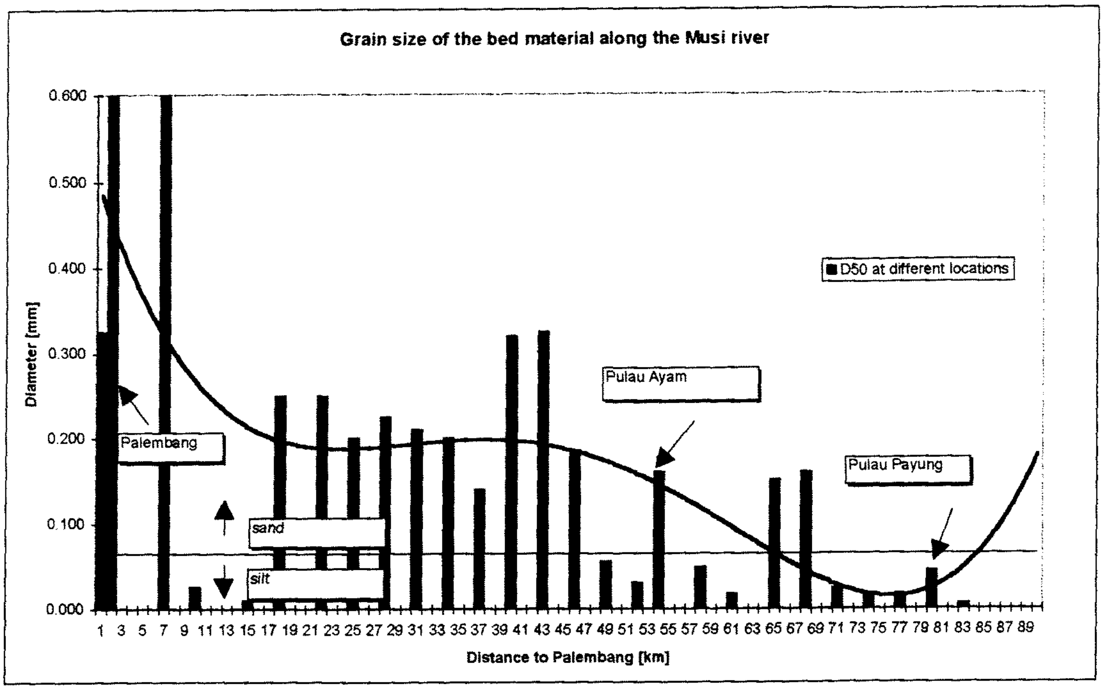

Figure 2—9 Grain size based on bottom samples.

The location where the samples where taken is in the middle of the river. The sampling method is not known.

2.2.5 Salinity

The Haskoning report [10] mentions that the approximate penetration of salt in a very dry season reaches the Delta Upang (confluence of the Musi and the Upang rivers). In the Euroconsult report [9] results of the salinity periodic observation (in the dry and wet season of 1994) show that the saline intrusion during dry season reaches Delta Upang with a salinity of 1 ‰, which is comparable to the Haskoning report. During the wet season the saline intrusion reaches only as far as Payung Island, close to the mouth of the Musi river. At high water slack during the dry season of 1994, the saline intrusion is maximum, which indicates that the saline intrusion can reach Delta Upang and Selat Jaran.

During the measurement campaign in January 1996, salinity samples have been taken at 5 locations during the spring and neap tide, i.e.:

- Sungsang
- Pulau Ayam
- Selat Jaran
- Sungai Lais
- Port of Palembang

During the spring tide the salinity of the Musi river at the Port of Palembang amounts to around 0.3 %; and during neap it is 0.2 %. It seems the salinity does not change in the same manner as the water level. The saline intrusion can not be determined on the basis of the field measurements, but has to be determined by periodic observations which is related to the fresh water discharge of the Musi river.

2.2.6 Conclusions on basis of the measurements

- Since in July a complete series of measurements of the water levels, velocities in the Musi river is available, it was decided that the July spring-tide measurements will be used for the calibration of the DUFLOW model of the Musi river (Chapter 3). This is because of the fact that for the calibration of a tidal propagation model, data of the tidal range, tidal phase and velocities should be available.
- The measurements results on the soil types correspond with the results of [10] and will be used in the sediment transport calculations (Chapter 4) and the dredging calculations (Chapter 6).
- The salinity measurements should be used for flocculation calculations. The saline intrusion can not be determined from the field measurement, it has to be determined by periodic observations which is related to the fresh water discharge of the Musi river

2.3 Siltation of the Musi river

In order to determine the optimal navigation depth to be dredged and maintained on the Musi river it is necessary to estimate the siltation rates at different depths and locations, to establish the required maintenance costs. Then these costs can be compared with the benefits which result from the reduction of the waiting times of the ships [3].

The sediment transport on the Musi river (and in general) can be classified according to origin and mechanism as in the following scheme [16]:

- *Bed load* (transport) is defined as the transport of bed material by rolling and sliding.
- *Suspended load* (transport) is defined as the transport of sediment which is suspended in the fluid for some time.

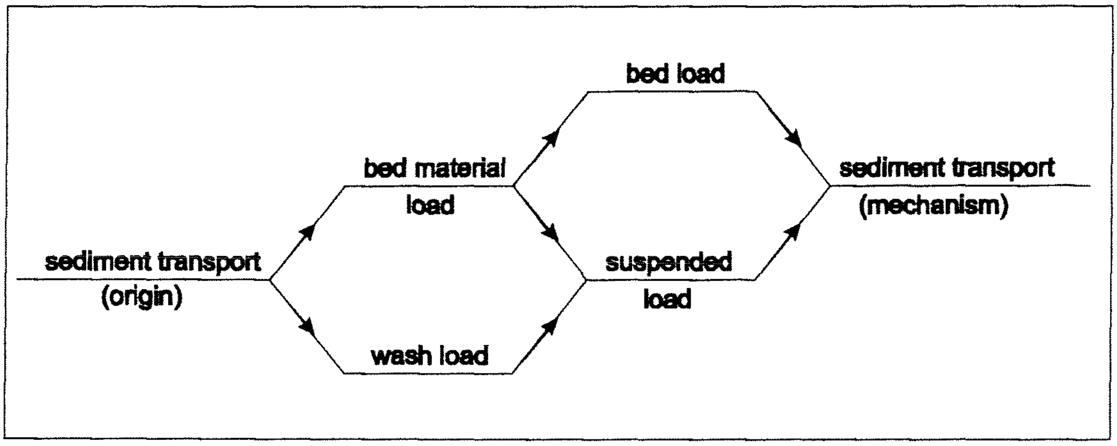

Figure 2—10 Classification of sediment transport.

- According to the mechanism of suspension, the suspended sediment may belong to the *bed material load* or the *wash load.*
- *Wash load* is defined as the transport of material finer than the bed material. It bears no relation to the transporting capacity of the stream; the rate is determined by the amount which becomes available by erosion in the catchment area upstream. Usually a diameter D with 50 $\mu m < D < 70$ μm is taken as a practical distinction between wash load and bed material load. As can be seen in Figure 2—9, the grain sizes of the bottom samples upstream of P. Ayam is larger, i.e. fine sand.

In the Haskoning report [10] it is mentioned that the Musi river has a yearly sediment transport in the order of 200,000 m^3/year, which is a small amount. It should be realized that this volume is an indication for the bed material load and that the total transport, which includes wash load, will be much higher.

The wash load transport is of little significance beyond the saline influence zone, as only the bed material load will take part in the morphological processes. However, when entering the saline area, the silt will gradually start settling and the bulk of the material will settle outside the river mouth forming a bar.

According to Frankel (1977) the coast near the Musi river accreted in a seaward direction with a propagation speed of 50 - 100 m/year, which would imply that Palembang was located on the coast about 700 years ago. To estimate the wash load, the following can be considered. If the extent of coast accreted by the Musi river depositions, is assumed as being at least 10 km wide with a height of 4 m, the amount of sediment deposited, would be 4 x 10,000 x (50 - 100) = 2 - 4 million m^3/year. Some of this sediment may of course come from other sources besides the Musi river, offshore or from other rivers (no data available).

In chapter 4 calculations will be made on the amount of sediments that will settle on the Musi river. Paragraph 4.2 will go into the sedimentation at the river shoals and paragraph 4.3 will discuss the sedimentation near the river mouth and at the outerbar.

2.4 Riverworks

In general a distinction can be made between short-term and long-term river improvements to influence the rivers characteristics or behavior, i.e. in this case the local water depth at certain points along the Musi river:

- short-term river improvements which have to be executed regularly (every one of two years), consisting mainly of dredging,
- long-term river improvements, which have a more permanent character. They consist of various kinds of structures like, groynes or in this case the closure of one branch around island (P. Payung).

When talking about 'riverworks' in this thesis 'long-term river improvements' are meant. At present there are no riverworks (besides dredging) in the Musi river.

The following two cases of riverworks will be viewed:

- Set 1 of riverworks: groynes at shoals along the Musi river to constrict the flow width and maintaining the river's depth through dredging.
- Set 2 of riverworks: groynes at shoals along the river + closure of one branch around P. Payung (island in the river mouth) and maintaining the rivers depth through dredging.

Chapter 5 will go in to the location and the involved costs of the two different alternatives.

2.5 Dredging

The navigation depth of the Musi river is at present maintained by dredging at about LWS -6.5 m. However near P. Payung and on the outerbar the depth is less as can be seen in Figure 2—4.

The dredging has for a number of years been carried out by the state owned dredging company Rukindo. Nearly 80 % of the dredging volume is carried out at the outerbar of the Musi river. In the last 10 years there was an annual average dredged volume of $V = 2.3 * 10^6$ m^3/a on the whole Musi river to maintain a depth of Local LWS -6.5. A specification of the dredged amounts can be found in the Appendix B.

Part of the river	(m^3/year)	%
Upstream Pulau Ayam	318,515	13
River mouth	210,935	9
Outerbar	1,863,724	78
Total	2,393,173	100

Table 2-2 Average dredging amount during the period '92-'95.

The volume is counted as a situ soil, according to Rukindo. They state they dredge good sand (see Figure 2—9) from Palembang up to Pulau Ayam for building projects in Palembang and silt at P. Payung and the outerbar. Paragraph 6.5.5 will go further into the amounts calculated by Rukindo.

Parties

Three parties are involved in the costs as a result of the waiting times and the costs of dredging work (See Table 2-3), because all three parties benefit from the better accessibility of the Port of Palembang.

- *Pelabuhan Indonesia II* is the harbor authority of Palembang, i.e. Indonesia Port Corporation II. They manage 12 ports in Sumatra, South-West Kalimantan and Eastern Java.

- *Pertamina* is the state oil company of Indonesia, i.e Indonesia's State Oil & Gas Company. They export crude oil from Sumatra via the Port of Palembang and the Musi river with large ocean-going vessels.
- *Pusri Companies* produces fertilizer and uses large ocean-going vessels for the transshipment of the mass-break bulk.

Parties	**Share of dredging costs**
Pelabuhan Indonesia. II	15 %
Pertamina	60 %
Pusri Companies	25 %

Table 2-3 Contributors to dredging costs.

Chapter 6 will go into the used dredging methods, calculation of the amount to be dredged and the involved costs, for different maintenance depths.

3. Musi River Model in DUFLOW

3.1 Introduction

Chapter 3 deals with the DUFLOW modelling of the Musi river (see Figure 1—2). The model has been developed in order to get a good insight into the parameters that control the Musi river. This is necessary for further study of the Musi river.

DUFLOW is a micro-computer software package for the simulation of one-dimensional unsteady flow and water quality in open channel network systems. The software used is the *DUFLOW version 2.04* with the *Manual edition 2.1, December 1995* [8].

Three Dutch institutes have cooperated in developing DUFLOW, viz.

- The International Institute for Hydraulic and Environmental Engineering (IHE), Delft.
- Rijkswaterstaat (Public Works Department), Tidal Waters Division, The Hague, The Netherlands.
- The Delft University of Technology, Faculty of Civil Engineering.

A DUFLOW model is well suited to investigate hydraulic changes due to river-based structures and improvement works. To make optimal use of the Musi river as an approach channel for the Port of Palembang the consequences of these changes have to be taken to account.

For every section (begin, middle, end) the DUFLOW output consists of:

- Water levels: h , [m]
- Discharges: Q , [m^3/s]
- Velocities: v ,[m/s]

The water levels are mainly of interest for the 'Vessel Traffic Study', while the discharges and velocities are important to the 'Dredging and River Study'.

First a brief description of DUFLOW will be given in paragraph 3.2.. Then paragraph 3.3 deals with the DUFLOW modelling of the Musi river. In paragraph 3.4 the model is calibrated with the available data and in paragraph 3.6 results of the calibration will be discussed. In paragraph 3.8 conclusions will be discussed. A complete review of the DUFLOW files is given in Appendix C.

3.2 Physical and mathematical background

DUFLOW is based on the one-dimensional partial differential equations that describe non-stationary flow in open channels [8]. These equations are a mathematical translation of the laws of conservation of mass and momentum, which read:

$$B\frac{\partial H}{\partial t}+\frac{\partial Q}{\partial x}=0$$

and

$$\frac{\partial Q}{\partial t}+gA\frac{\partial H}{\partial x}+\frac{\partial(\alpha Qv)}{\partial x}+\frac{g|Q|Q}{C^2AR}=B\gamma w^2\cos(\Phi-\phi)$$

while the relation

$$Q=v\cdot A$$

holds and where:

t time[s]

x distance as measured along the channel axis [m]

H(x,t) water level with respect to the reference level [m]

Q(x,t) discharge at location x and time t [m^3/s]

v(x,t) mean velocity (averaged over the cross-sectional area) [m/s]

R(x,H) hydraulic radius of cross-section [m]

A(x,H) cross-sectional flow area [m^2]

b(x,H) cross-sectional flow width [m]

B(x,H) cross-sectional storage width [m]

L cross-sectional length [m]

g acceleration due to gravity [m/s^2]

C(x,H) coefficient of Chézy [$m^{½}/s$]

$w(t)$ wind velocity [m/s]

$\Phi(t)$ wind direction [°]

$\phi(x)$ direction of channel axis, measured clockwise from the north [°]

$\gamma(x)$ wind conversion coefficient

α correction factor for non-uniformity of the velocity distribution in the advection term, defined as:

$$\alpha = \frac{A}{Q^2}\int\int v(y,z)^2\,dydz$$

where the integral is taken over the cross-sectional area A.

For the derivation of these equations it is assumed that the fluid is well-mixed and hence the density may be considered constant.

3.2.1 Discretization of the unsteady flow equations

These equations have been discretized using the 4-point implicit Preissman [8], [22] scheme (see also Figure 3—1).

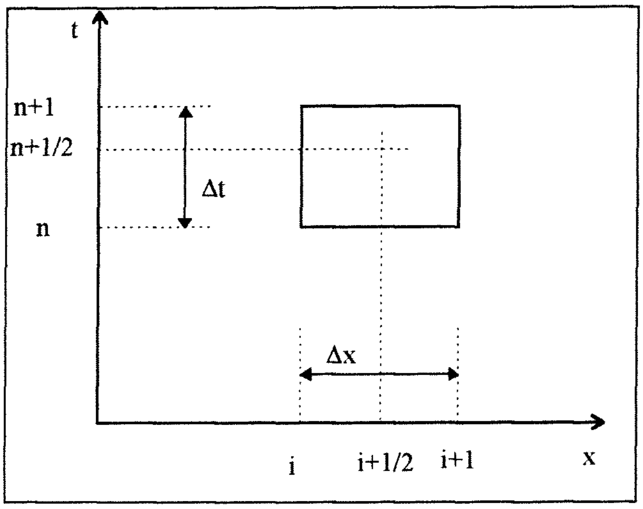

Figure 3—1 The four point Preissman scheme.

Finally, for all the network sections (see Figure 3—6) two equations are formed, one is a discretized version of the continuity equation the other is a discretized version of the equation of motion. Unknowns on the new time level, t^{n+1}, are the discharge, Q, and water level, H, with coefficients, N:

$$Q_i^{n+1} = N_{11}H_i^{n+1} + N_{12}H_{i+1}^{n+1} + N_{13}$$
$$Q_{i+1}^{n+1} = N_{21}H_i^{n+1} + N_{22}H_{i+1}^{n+1} + N_{23}$$

3.2.2 Boundary and initial conditions

For a unique solution of the set of equations additional conditions have to be specified at the physical boundaries of the network. These conditions can for instance be a (tidal) elevation H(t) or a discharge Q(t). At internal junctions the (implicit) condition states that the water level is continuous over such a junction node.

To start the computations, initial values of H and Q are required for each section, although the influence will disappear in time. These values can be historical measurements, results of from former computations or just a first educated guess.

3.2.3 Practical considerations

1. Extrapolation of simulation results to new situations should be done carefully. A model might give an outstanding description of the present situation after calibration, but due to changing circumstances in the new situation the relative importance of certain processes may change.
   The boundary conditions in particular must be chosen with great care in cases where a change in the system may affect a boundary condition, which in turn may influence the hydraulic conditions in the region of interest. Since the same boundary conditions are applied in the present and in new situations, this may lead to erroneous results in the simulation of future changes. It is therefore important to ascertain that:

   Any change in the system does not affect the state at a boundary condition

2. The best type of boundary condition to be used is that quantity or relation that is the least sensitive to the state in the model itself.
   Therefore the upstream boundary condition in a river is preferably a discharge, whereas the downstream boundary condition should be a water level, if the river flows into a lake or sea.

3. Very detailed schematization of a network is seldom necessary due to the nature of the equations involved. Small changes in cross-sections usually have only a slight influence on the state in a region of interest.

3.2.4 Limitations

1. The equations are designed for 1-dimensional flow. This means that a current with significantly changing velocity profiles in the vertical cannot be modeled. For instance, the model is not suitable for stratified waters. Moreover, the flow has to be directed roughly parallel to the channel axis. The strong tidal motion on the Musi river is a reason why the velocity measurements and the DUFLOW computations differ.
2. The water density is assumed to be constant.
3. The numerical solution method is not valid for supercritical flow in open channels.

3.3 Modelling of the Musi River Model

3.3.1 Layout

The Musi river model is designed to reproduce the water levels, discharges and velocities in the Musi river at locations ranging from Palembang to the outerbar. For this the river has been divided into a number of sections. These sections together form a network [see Figure 3—6]. Each section has its own characteristics and they must be categorized according to the available river data.

Besides the Musi river, the model has to consist of 3 branches:

- The Sungai Upang
- The Selat Jaran
- The Sungai Telang

To be able to run the model, the boundary conditions have to be formulated. The boundaries should be selected at locations where more or less independent conditions hold. In this case a tidal water level variation at the sea side of the model and the fresh water discharge upstream from Palembang were used. The latter boundary should be located where there is little or non tidal influence.

From the measurements carried out by ITS it is clear that there is hardly any loss of tidal influence at Palembang (see Figure 2—3). Therefore the upstream boundary of the model should be at least 70 km upstream from Palembang. This results in the following schematic representation (see Figure 3—2) of the layout of the model upstream of Palembang:

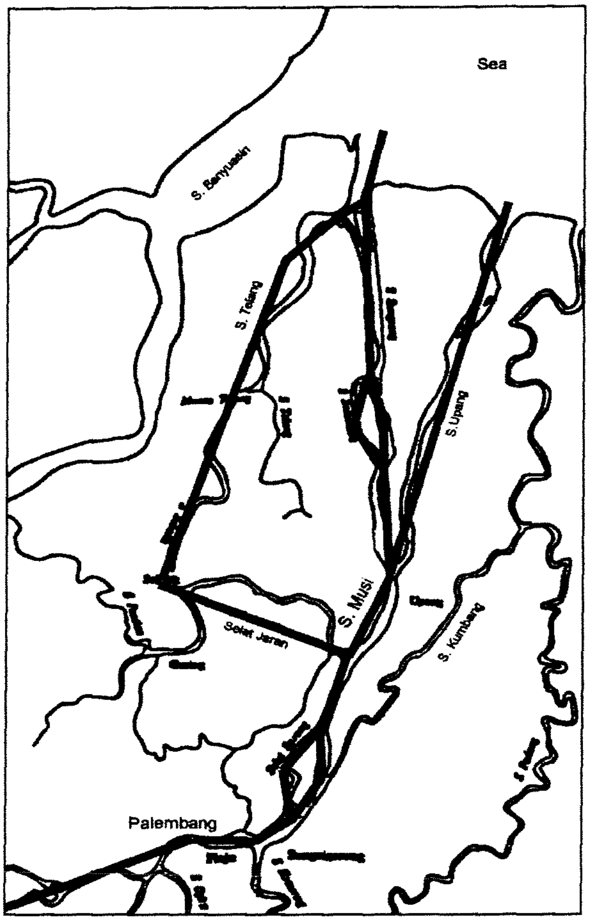

Figure 3—2 Layout of the DUFLOW model for the Musi river.

3.3.2 Physical characteristics

3.3.2.1 Longitudinal profile

The local depths are read from [7] and implemented in the DUFLOW model of the Musi river after correction to account for the average bottom slope (see Figure 2—4).

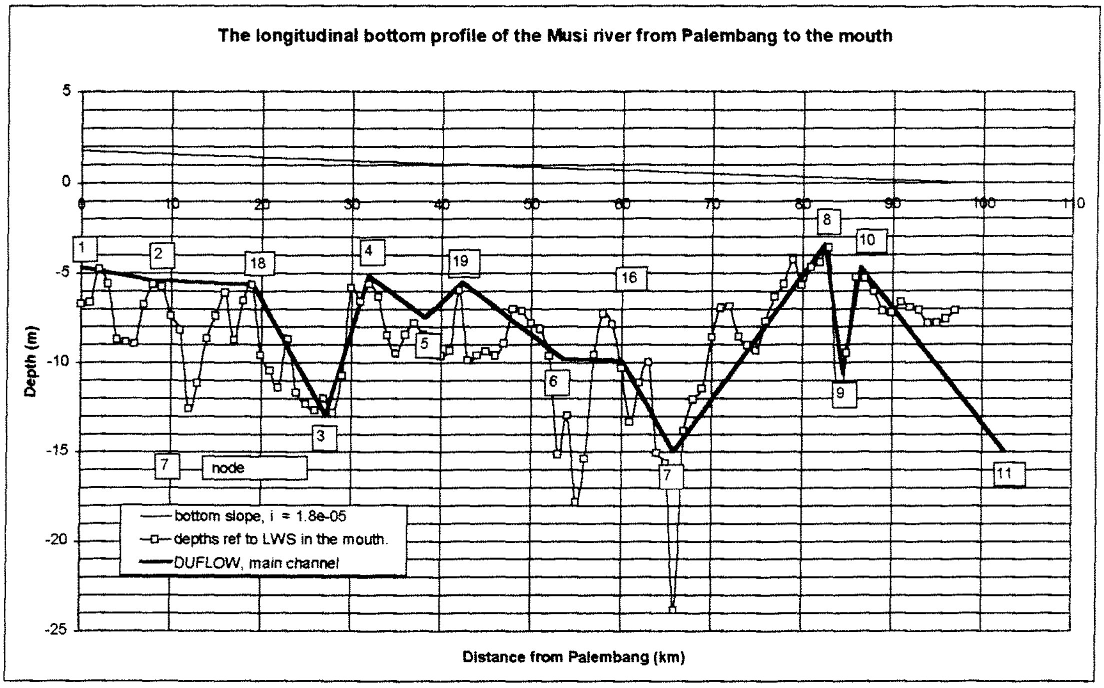

Figure 3—3 The longitudinal bottom profile of the Musi river navigation channel and its implementation in the DUFLOW model of the Musi river.

3.3.2.2 Cross sections

One aspect of making a DUFLOW model of a river is the schematizing of the cross sections of every river section. As a first approach it is assumed that the cross sections of the Musi river have the same storage width and flow width for each water level (see Figure 3—4):

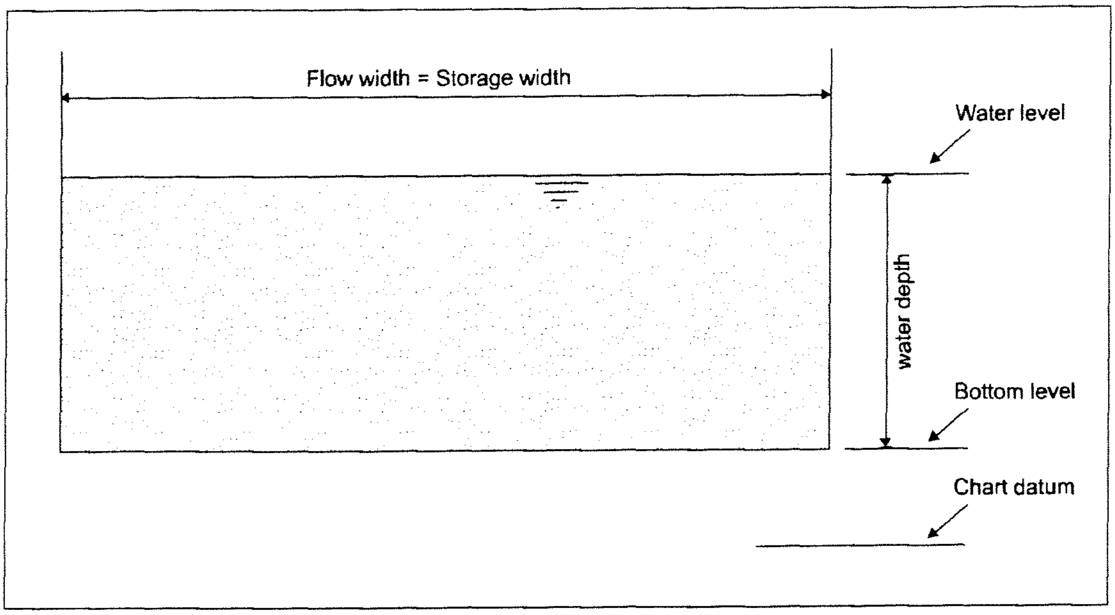

Figure 3—4 First approach to cross section implementation.

The consequence of this approach will be that the velocity profiles produced by the model will deviate (little) from the velocity profiles encountered in reality. When a more detailed model is developed this can be corrected by applying the bathymetry measurements, although as a result of the proportions of the cross section (see Figure 3—5) the influence will be minor.

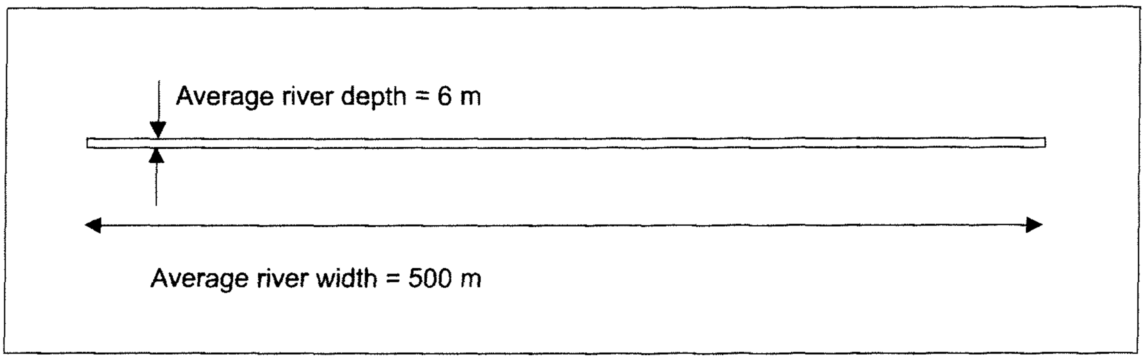

Figure 3—5 Cross section of the Musi river with average depth and width in proportion.

The flow and storage width and the depth of the cross-sections of the Musi river model can be derived from the results of the measurement campaigns carried out, in connection with this study in January and July 1996 [15], from previously conducted studies on the Musi river [10, 17] and from a navigation map [7].

3.3.2.3 Chézy values

According to the measurement results there is a phase difference of about 5 hours between the tidal motion in the mouth of the Musi river and the tidal motion near Palembang. This indicates[1] that the average Chézy value should lie between 45 and 60 [$m^{½}/s$]. In a verification calculation the Chézy coefficient can be expressed as follows.

If the following values are assumed,

Q = 1500 [m^3/s]

B = 500 [m]

d = 6.0 [m]

i = $1.8 * 10^{-5}$ [m]

and

$$C = \frac{Q}{Bd^{\frac{3}{2}}i^{\frac{1}{2}}} = 50\ [\ m^{½}/s]$$

it seems to be that a first approach of C = 50 [$m^{½}/s$] will be a reasonable estimation.

3.3.3 Nodes and sections

From the cross sections the widths are estimated at several depths along the Musi river. When there is a significant change in water depth or width a new section and therefore a node needs to be implemented.

At the confluences and bifurcations there is a change in the discharge of the main channel (ΔQ) and therefore there must be a node at these places. Of course there is also a need to define the boundary conditions of the model at the sea and upstream boundaries.

The model will be used to get information on the water levels, water velocities and water discharges as function of time at several locations. One of the locations will be Pulau Payung, at which the closure of one of the two in parallel channels around P. Payung has to be considered in this study (see paragraph 3.7). Therefore the two in parallel channels around Pulau Payung will be implemented in the model. Further upstream, around Pulau Ayam and Selat Borang also 2 channels are implemented.

In the next figure data of the sections is given to get more insight in the network. The depth at begin and end of the sections are referred to the reference level of the model (see Figure 3-4, page 34).

Section number	begin node	end node	length (m)	depth begin (m)	depth end (m)
1	1	2	7679	-4.70	-5.41
2	2	17	11931	-3.68	-3.76
3	3	4	8003	-12.93	-5.25
4	4	5	10975	-5.25	-7.46
5	5	19	5025	-7.46	-5.48
6	6	15	10574	-9.73	-2.31
7	7	8	12291	-14.98	-3.43
8	8	9	4697	-3.43	-10.62
9	9	10	2462	-10.62	-4.67
10	10	11	20579	-4.67	-15.00
11	8	24	3288	-2.77	-4.50
12	4	12	5411	-6.00	-6.00
13	12	13	45000	-4.50	-4.00
14	13	9	10262	-4.00	-4.00
15	15	7	5506	-2.31	-9.69
16	6	16	5025	-9.79	-9.89
17	16	7	5772	-9.89	-14.98
18	17	3	7621	-3.76	-6.26
19	2	18	5396	-5.41	-5.66
20	18	3	8150	-5.66	-12.93
21	19	6	6519	-5.48	-9.79
22	25	23	34184	-5.50	-6.00
23	20	21	7071	7.00	6.50
24	21	22	14142	-2.50	-3.50
25	22	33	14142	-2.50	-2.94
26	23	14	6235	-6.00	-6.50
27	24	10	3288	-4.50	-5.86
28	5	25	5050	-5.00	-5.50
30	30	1	14142	-4.26	-4.70
31	31	30	14142	-3.82	-4.26
32	32	31	14142	-3.38	-3.82
33	33	32	14142	-2.94	-3.38
40	21	41	21213	5.50	5.50
41	40	42	21213	4.24	4.25
42	41	43	21213	3.00	3.00
43	42	44	21213	1.75	1.75
44	43	45	21213	0.50	0.50
45	44	46	21213	-0.75	-0.75
46	45	47	21213	-1.50	-1.50
47	46	48	21213	-2.00	-2.00
48	47	22	14142	--2.50	-2.50

Table 0-1 Network data.

The implementation of the *sections* is now straight forward: just follow the river (from the upstream boundary to the downstream boundaries) and at each node, a new section starts.

The above mentioned procedure results in the following network (see Figure 3—6) of the Musi river; the definition of the nodes, sections and cross sections can be found in the Appendix C:

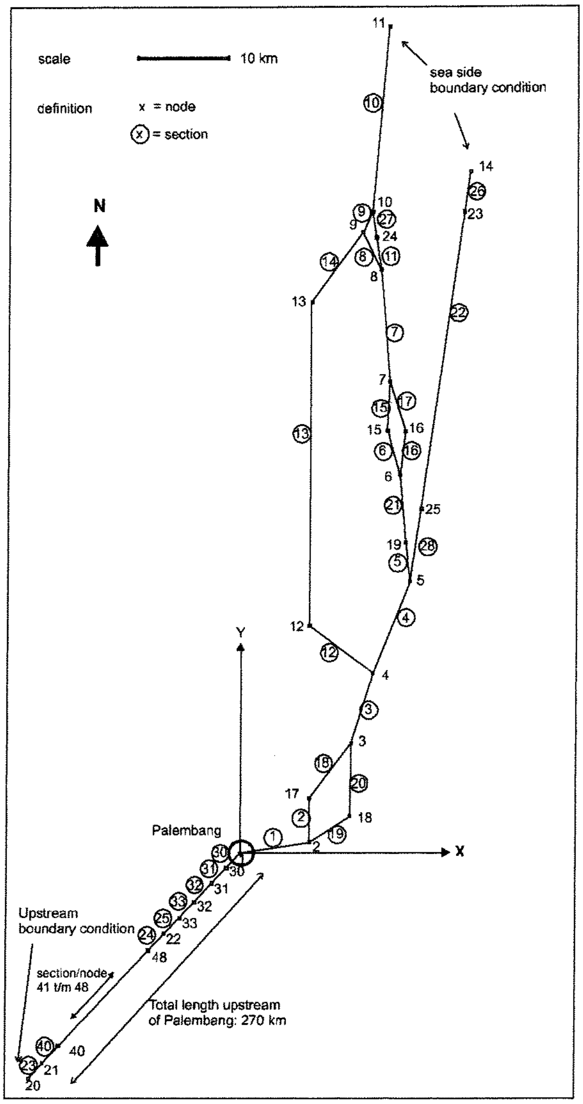

Figure 3—6 Network of the Musi river DUFLOW model.

[1] The friction of the river bed largely determines the resulting propagation speed of the tidal wave.

3.3.4 Initial conditions and boundary conditions

3.3.4.1 Initial conditions

At each node in the model initial conditions at the start of the computation have to be defined. However, if the boundary conditions are selected correctly, the initial conditions are not very important, since their influence will be limited to the first couple of hours in the DUFLOW computation.

Initial condition	value
water level[1]	1.90 m
water discharge	0 m^3/s

Table 3- 3-1 The initial conditions at each node for the DUFLOW computations.

From the start of the calculations the initial conditions are dominating the results over several hours. The results are useful when these influences have been damped out by friction.

3.3.4.2 Start of the computation

As mentioned in the paragraph 3.3.4.1 the damping of the initial conditions will take a few hours. The measurements have been carried out during two days from July 29th 1996 to July 31st 1996 (at spring tide during dry season). The simulation will start one day earlier (three days, July 28th until July 31st) until the initial conditions have minor effect. It is now possible to compare a two day simulation with the measurements (July 29th until July 31st).

3.3.4.3 Boundary conditions

The boundary conditions of the Musi River Model are very important, since they control the behavior of the system once the influence of the initial conditions has disappeared. At each physical boundary of the model boundary conditions should be specified.

[1] The water level with respect to the reference level, i.e. LWS at sea, is the same at each node in the beginning of the computation.

Upstream boundary

At the upstream boundary of the system a fresh water discharge is specified. At first this is the dry season discharge (July) of the Musi river, $Q_{upstream} = 300\ m^3/s$. To reduce the influence of the downstream tidal motion on the most upstream section of the model it was necessary to define the upstream boundary condition 200 km upstream of Palembang (node 22, see Figure 3—6). As can be seen in Figure 3—7 the resulting upstream discharge has only little deviations from the constant level of 300 m³/s.

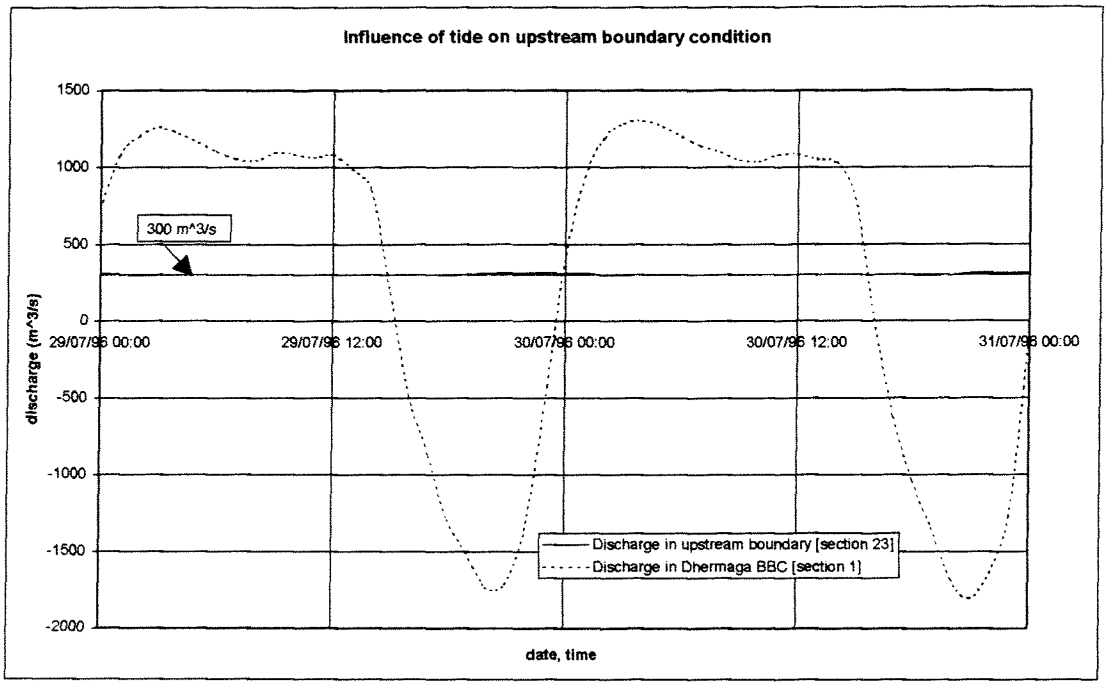

Figure 3—7 The upstream boundary conditions of the model.

Downstream boundary

At the downstream boundaries of the Sungai Musi and Sungai Upang (node 11 and node 14, see Figure 3—6), the tidal fluctuation in the open sea [6] is used (see Figure 3—8).

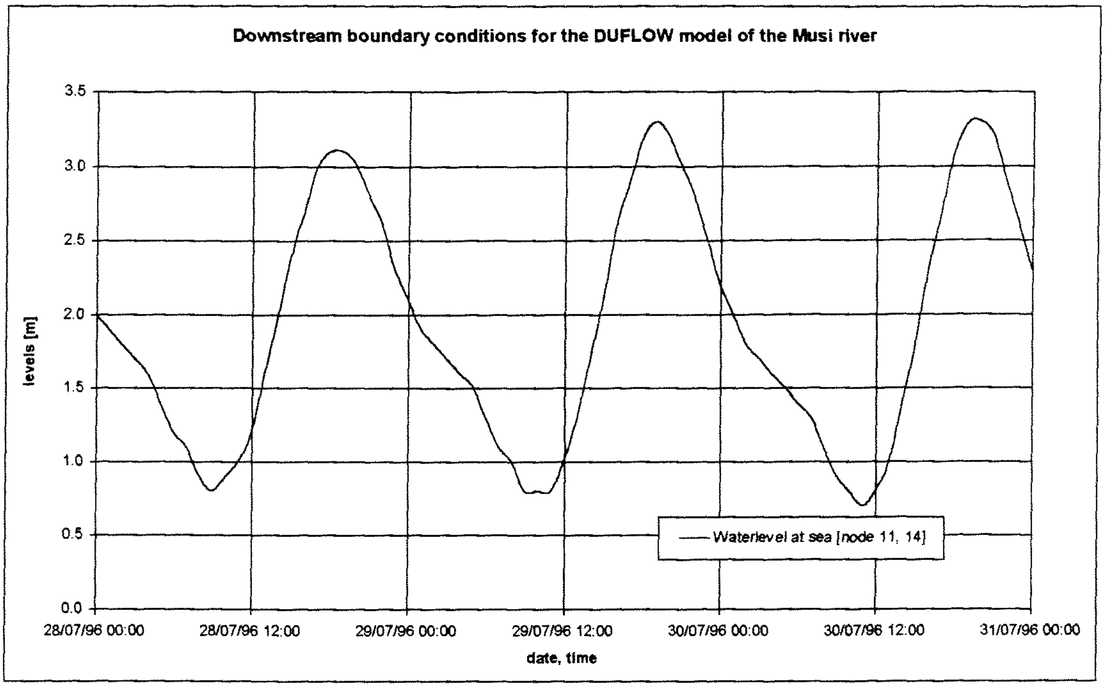

Figure 3—8 The downstream boundary conditions at the sea side of the model, e.g. at the mouth of the Musi and at mouth of the Upang river.

The range of the levels (see Figure 3—9 and Figure 3—10) at the downstream boundary compared with the measurements, are a little to small. A possible reason for this is the distance of 65 km between the location where the levels of the tide tables are given and the actual location of the Musi river mouth. It is possible that the range of the tidal levels increase while traveling over the coastal zone of 65 km, but on this no data is available.

3.4 Calibration of the Musi River Model

The model will have to be tested and calibrated. For this purpose the July 1996 water level measurements and water velocity measurements are used. Before using these measurements, they need to be critically examined (see chapter 2).

As mentioned in (paragraph 2.2.6) the spring tide measurements of July 1996 will be used to calibrate the model. From these measurements, only the following data can be used for calibration:

- Tidal range
- Tidal phase

- Velocities

Velocities are only used for a calibration of the velocity range, because the velocities are probably more accurate than the discharges. To calculate the discharges the cross section area has to be estimated, which is another possible source of error.

The model is calibrated according to the following steps:

First a reference run is made. In this run a first estimation for the upstream discharge (Q_0) of 300 m^3/s is used. This is followed by a series of sensitivity runs, in order to determine the parameters that influence the Musi river system at the most.

The results of these runs are compared with the measurements at 4 points,

- Dhermaga BBC
- Selat Jaran
- Pulau Ayam
- P. Payung

In Figure 3—9 to Figure 3—12 the results of the DUFLOW simulations together with the measurements of the water levels and the water velocities at several locations along the river are presented in one graph. As can be seen in Figure 3—11 and Figure 3—12 the velocity measurements can only be used for a calibration on the velocity range.

In the figures Figure 3—13 to Figure 3—20 the results of the DUFLOW computations after calibration are presented per location. First the water level calculations and then the velocity calculations. The best overall fit, for spring tide measurements in July, is obtained by using the following input, together with the previously mentioned data:

Q_0 = 300 m^3/s

i = $1.8e^{-5}$

C = 50 $m^{1/2}/s$

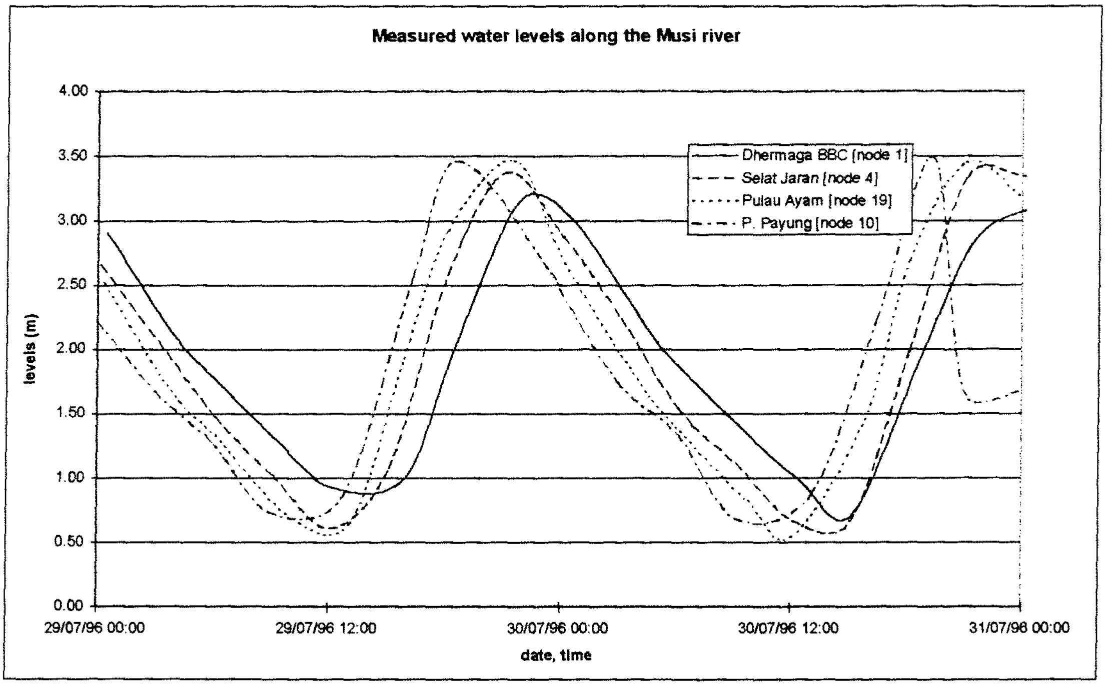

Figure 3—9 Measured water levels along the Musi river during spring tide in the dry season.

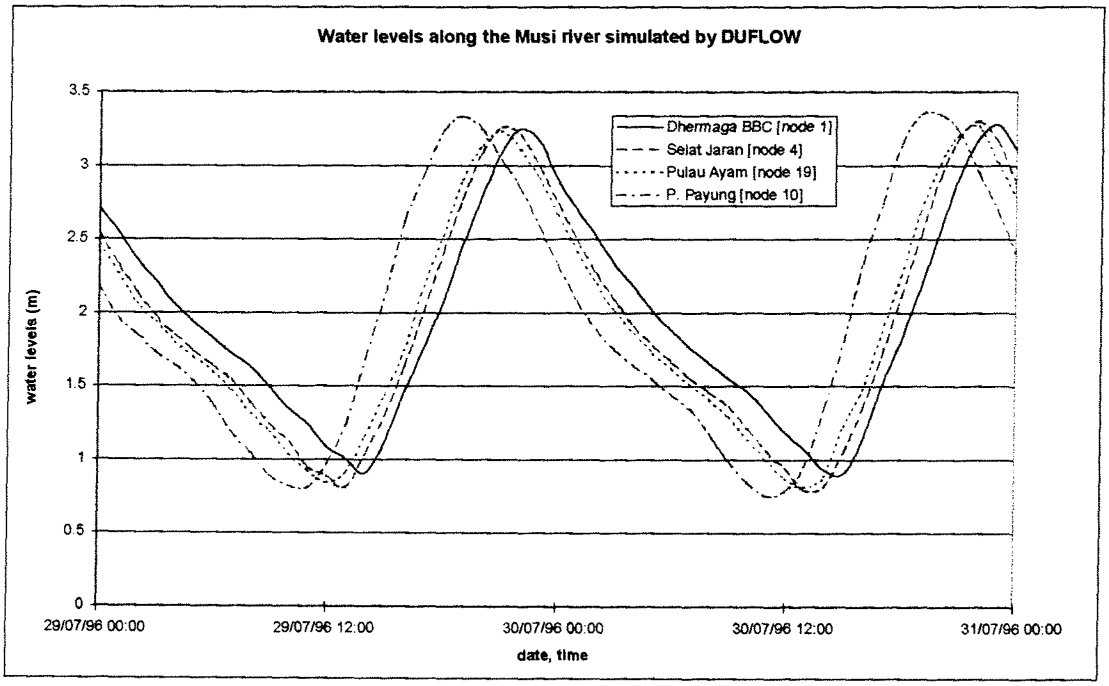

Figure 3—10 Water levels simulated by DUFLOW along the Musi river during spring tide in the dry season.

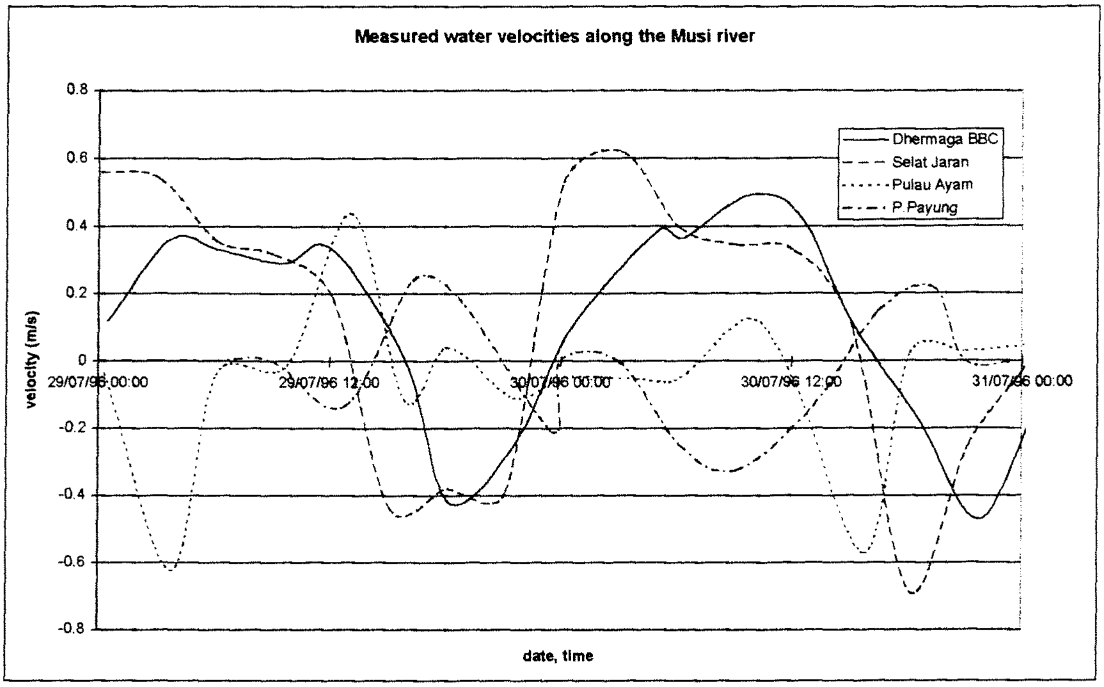

Figure 3—11 Measured water velocities along the Musi river at different locations during spring tide in the dry season.

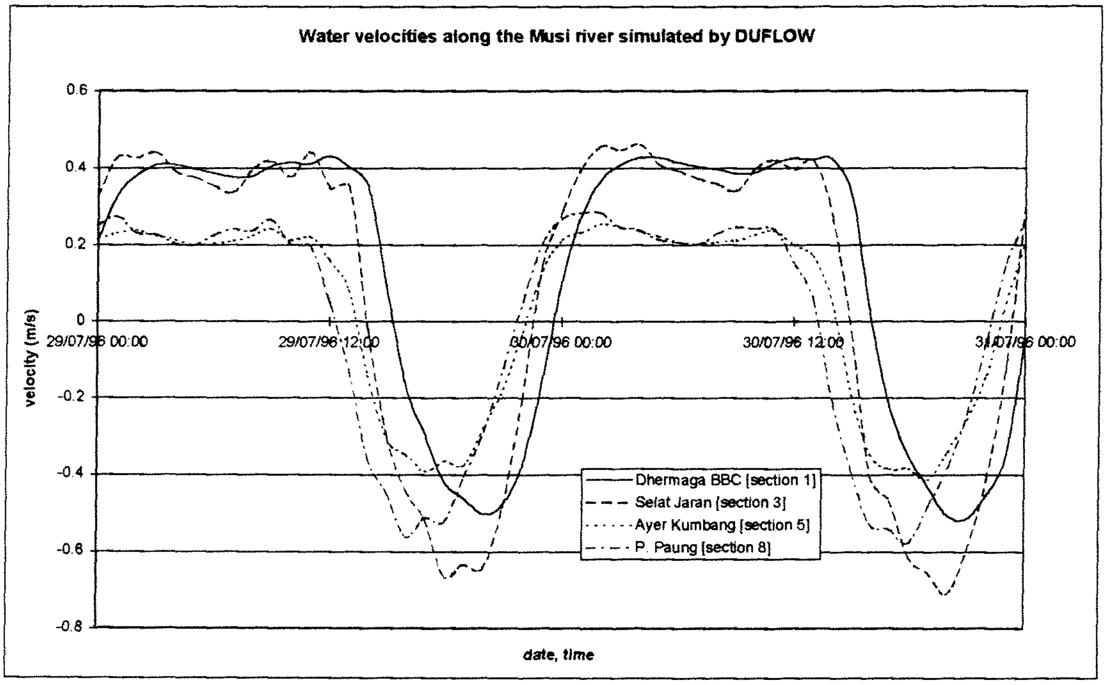

Figure 3—12 Water velocities simulated by DUFLOW along the Musi river during spring tide in the dry season.

From Figure 3—13 to Figure 3—20 can be seen that at other locations some concessions have been made, but this is to obtain the best overall fit. To make a better model of the Musi river and it's tributaries and bifurcations, more data and especially more accurate data is necessary.

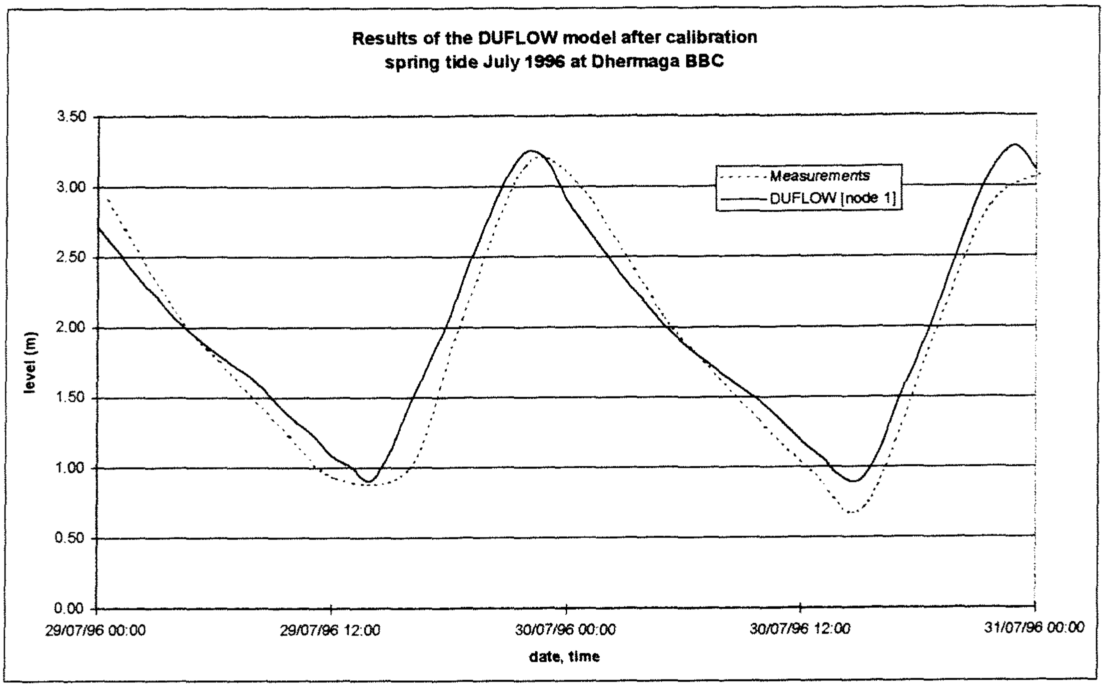

Figure 3—13 Water levels in dry season during spring tide at Dhermaga BBC.

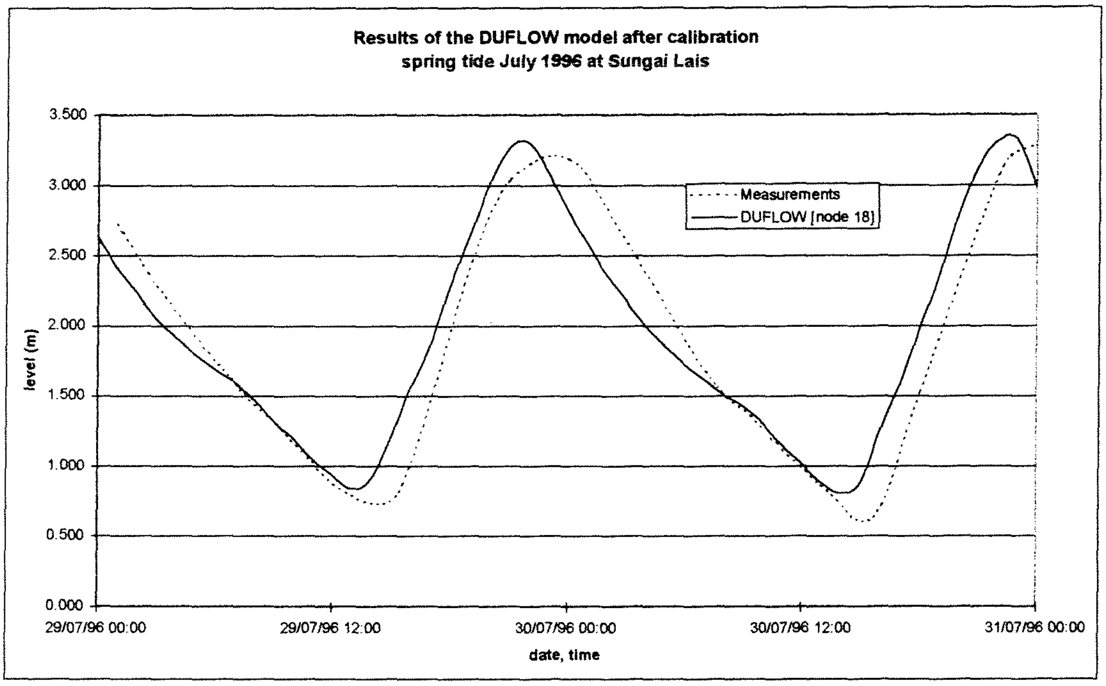

Figure 3—14 Water levels in dry season during spring tide at Sungai Lais.

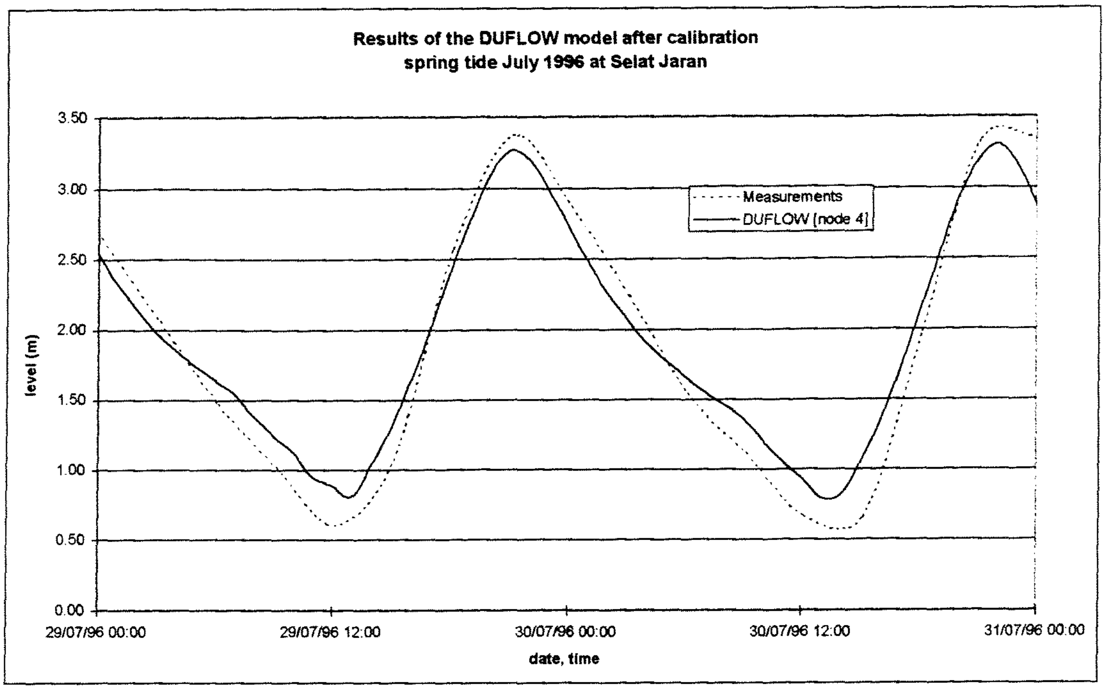

Figure 3—15 Water levels in dry season during spring tide at Selat Jaran.

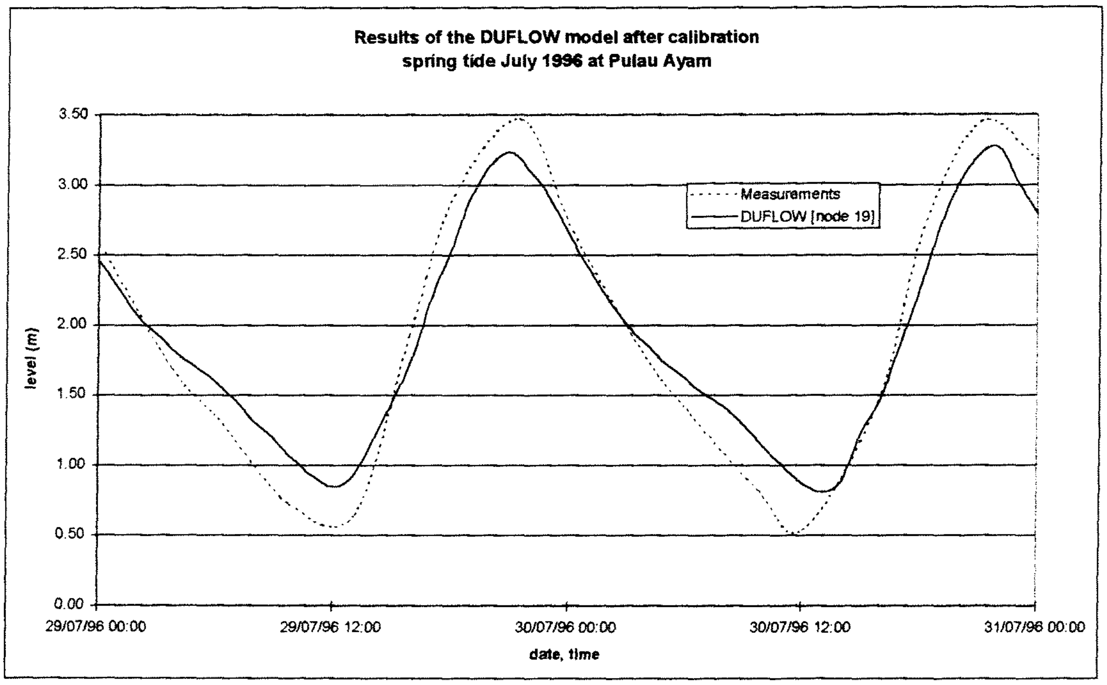

Figure 3—16 Water levels in dry season during spring tide at Pulau Ayam.

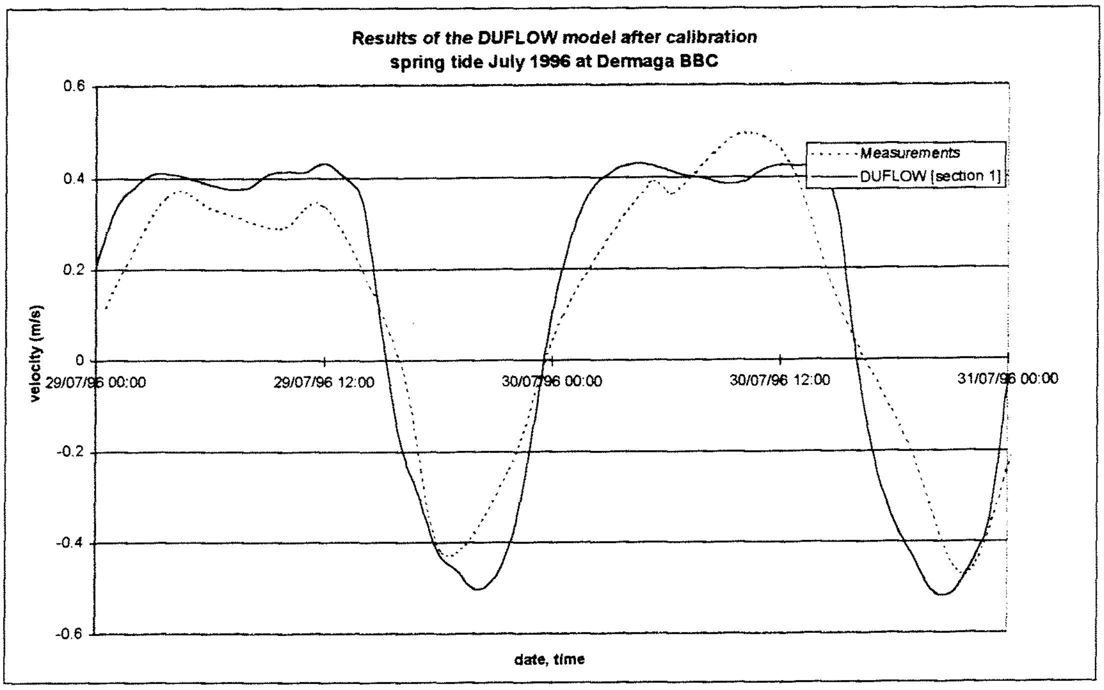

Figure 3—17 Velocities in dry season during spring tide at Dermaga BBC.

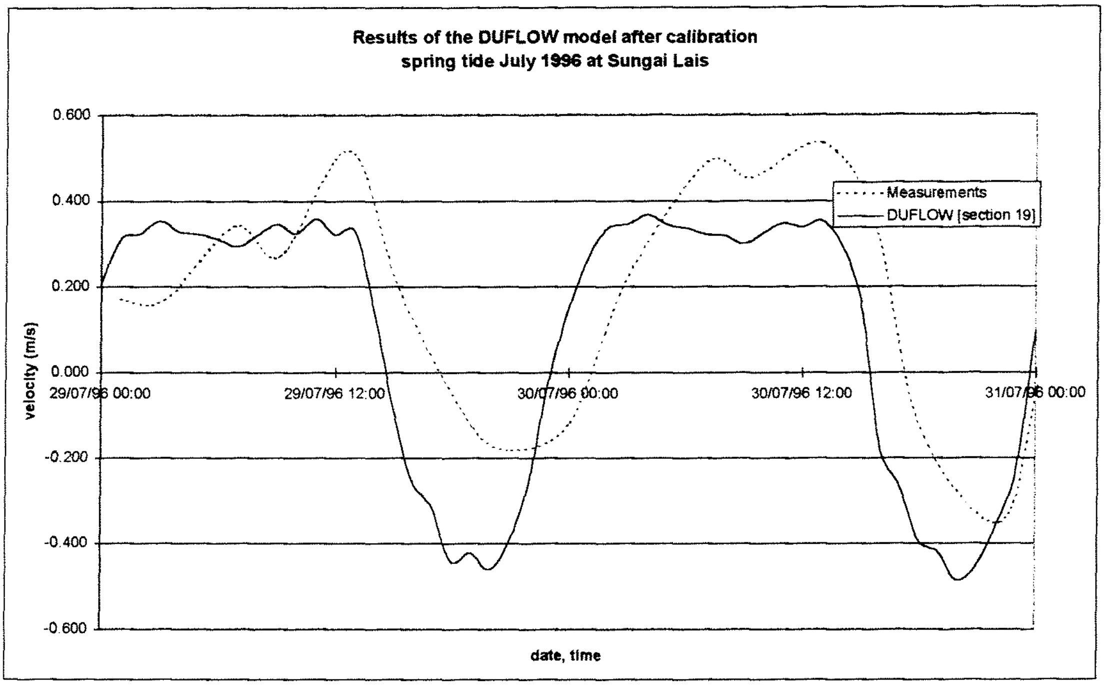

Figure 3—18 Velocities in dry season during spring tide at Sungai Lais.

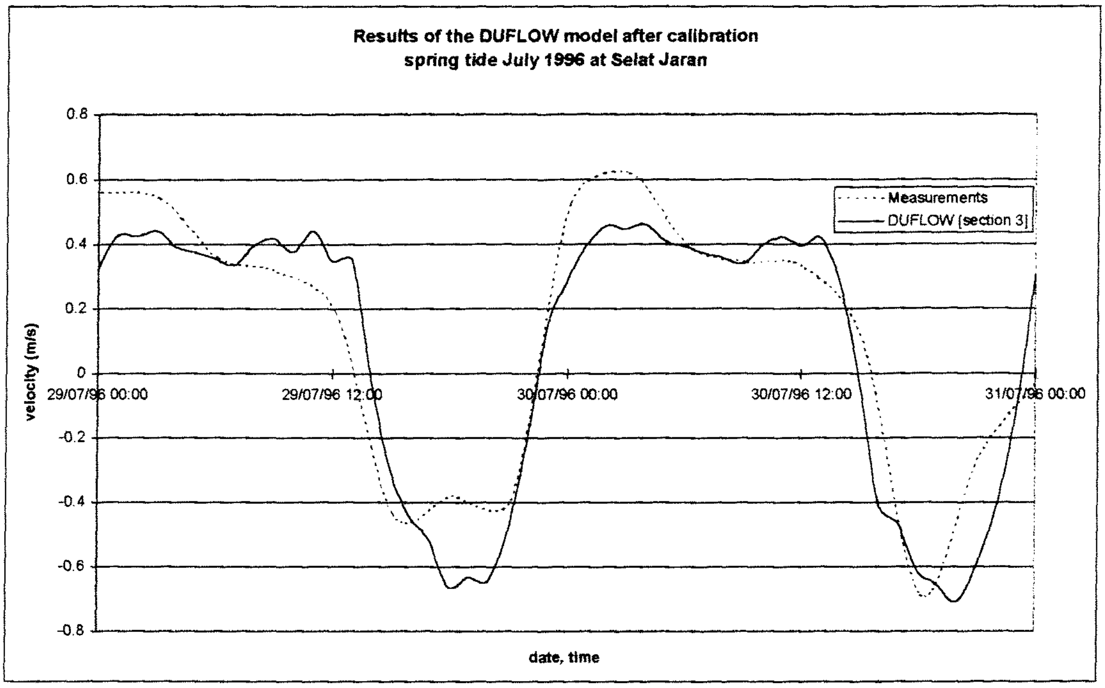

Figure 3—19 Velocities in dry season during spring tide at Selat Jaran.

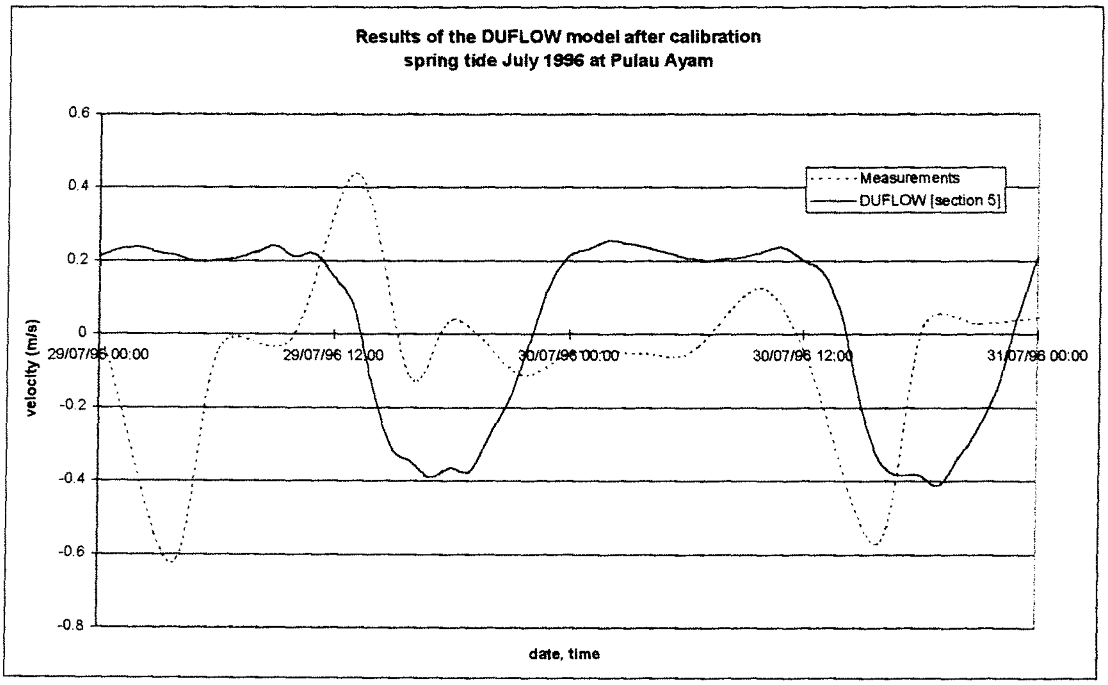

Figure 3—20 Velocities in dry season during spring tide at Pulau Ayam.

It should be noted that the model of the Musi river could *only* be tested for the *tidal range* and *tidal phase* of mainly the levels during the dry season (July 1996), since there was not enough information available from the measurements or any other source to recalibrate the model for the wet season.

During the wet season, the upstream discharge will be significantly higher than during the dry season. The influence on the total cross sectional discharge is not so large (see Figure 3—25). After careful consideration, the freshwater discharge during the wet season, at least during the January 22-23 1996 spring tide period, is estimated[1] at 700 m^3/s.

3.5 Verification of DUFLOW calculation

The following calculation is made to validate the results from DUFLOW.

Because of the long channel length in relation to the channel width the long wave problem can be reduced to a one dimensional problem. The propagation of a long wave (tidal wave) in a open channel is described with an equation of continuity (see equation (3-1)) and an equation of motion (see equation (3-2)).

$$B\frac{\delta h}{\delta t}+\frac{\delta Q}{\delta x}=0 \tag{3-1}$$

$$\frac{\delta Q}{\delta t}+\frac{\delta}{\delta x}\frac{Q^2}{A_s}+gA_s\frac{\delta h}{\delta x}+g\frac{Q|Q|}{C^2A_sR}=0 \tag{3-2}$$

The terms of equation (3-2) are divided by g^*A_s and both differential equations are integrated over the section length. When section average values are used a subscript *g* is added:

$$Q(x_2)-Q(x_1)=-B\Delta x\frac{dh_g}{dt}=-\Delta F\frac{dh_g}{dt} \tag{3-3}$$

$$h(x_2)-h(x_1)=-\frac{1}{gA_s}\left[\frac{dQ_g}{dt}\Delta x+Qu\big|_{x2}-Qu\big|_{x1}\right]-\Delta\frac{x}{C^2A_s{}^2R}Q_g|Q_g| \tag{3-4}$$

In this way the quantities related to the cross section area are made independent of x over the section length. These equations can be used to validate the DUFLOW calculations.

Equation (3-3) expresses for every point of time (t) that the discharge difference between the section boundaries equals the storage per unit of time in the section. Equation (3-4) expresses for every point of time (t) that the fall over the section length Δx equals the sum of the contributions of the following terms:

- Local inertia
- Advective inertia
- Resistance

The following equation results when equation (3-3) is substituted in the advective term of equation (3-4):

$$h(x_2) - h(x_1) = -\frac{1}{gA_s}\left[\frac{dQ}{dt}\right]_g \Delta x + \frac{2B}{gA_s^{\,2}} Q_g \left[\frac{dh}{dt}\right]_g \Delta x - \frac{Q_g|Q_g|}{C^2 A_s^{\,2} R}\Delta x \qquad (3\text{-}5)$$

where:

- Local inertia term is:

$$-\frac{1}{gA_s}\left[\frac{dQ}{dt}\right]_g \Delta x$$

- Advective inertia term is:

$$+\frac{2B}{gA_s} Q_g \left[\frac{dh}{dt}\right]_g \Delta x$$

- Resistance term is:

$$-\frac{Q_g|Q_g|}{C^2 A_s^{\,2} R}\Delta x$$

With the calculation of these terms it is possible to determine the influence of each term on the fall over the considered section. The functions of h(x,t) and Q(x,t) obtained from DUFLOW will be used in the verification calculation.

[1] Wet season in this case represents the circumstances that occurred during January 22-23 1996. To the author's knowledge there is no information available on the exact conditions during that period. Dry season represents the circumstances that occurred during the last days (29-30) of July 1996. To the author's knowledge there is also no information available on what these conditions were.

Goal of the verification calculation is to determine the influence of the different terms for a certain point of time and in what way will be meet on the differential equations. The calculations are carried out for a number of sections (see Figure 3—21 to Figure 3—24) at a certain point of time (time = 55; the verification should be valid for every point of time). The results are presented in the following table.

section	dQ/dt (m^3/s^2)	Qaver (m^3/s)	resistance term	local inertia	advective term	h (end) - h (begin) in (m)		
						manual	Duflow	difference
25	-0.017	388	-0.247	-0.006	-0.006	-0.260	-0.260	0.000
1	-0.032	1132	-0.082	0.003	-0.001	-0.080	-0.079	0.001
3	-0.031	1639	-0.043	0.005	-0.011	-0.049	-0.053	0.004
5	-0.022	790	-0.010	0.000	-0.002	-0.011	-0.012	0.001
7	-0.038	1124	-0.101	0.004	-0.002	-0.098	-0.153	0.055
8	-0.019	904	-0.009	0.000	0.006	-0.003	-0.003	0.000

Table 3-2 Results of DUFLOW verification calculation.

The difference between the manual calculated fall and the measured (DUFLOW) fall is in the order of one centimeter, which is acceptable.

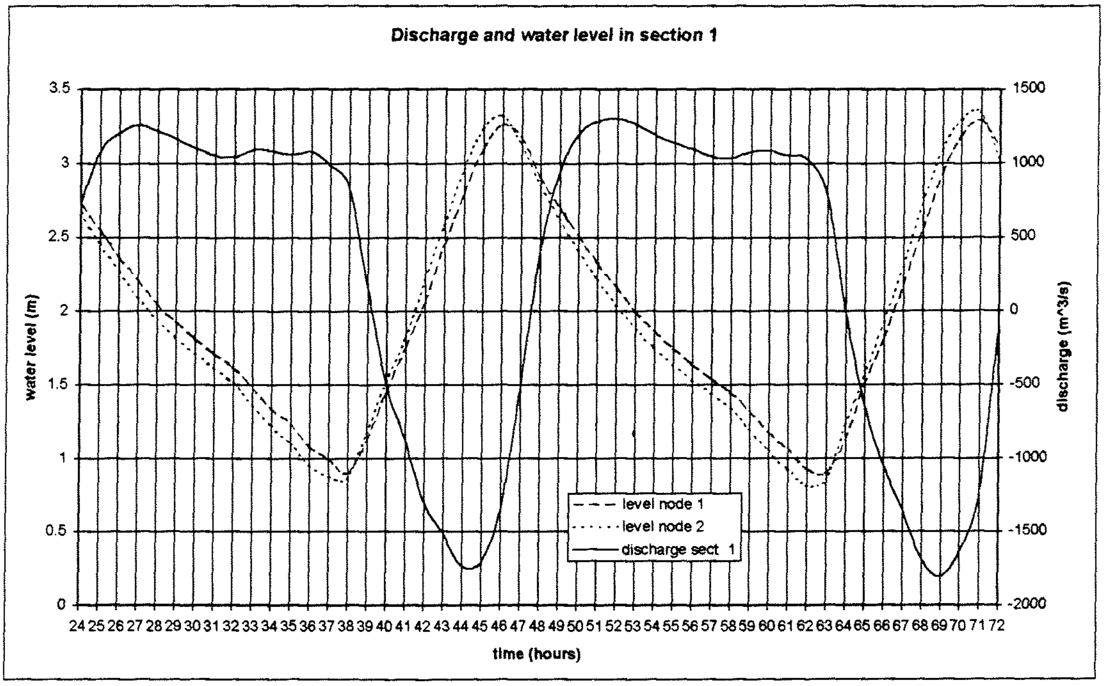

Figure 3—21 Discharge and water levels in section 1.

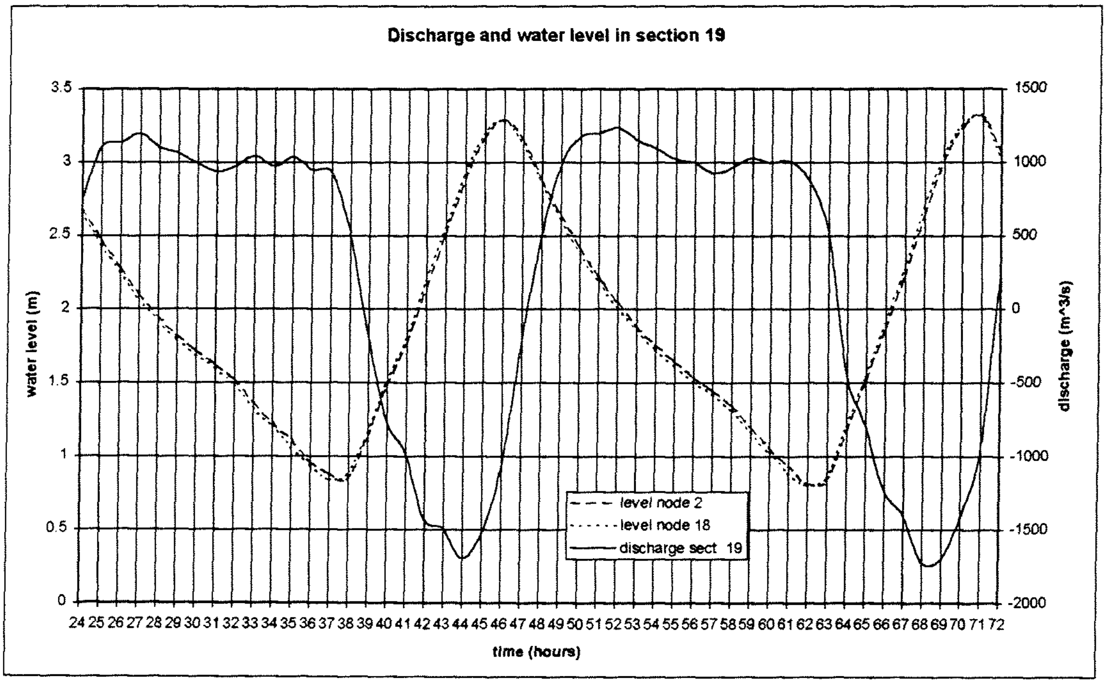

Figure 3—22 Discharge and water levels in section 19.

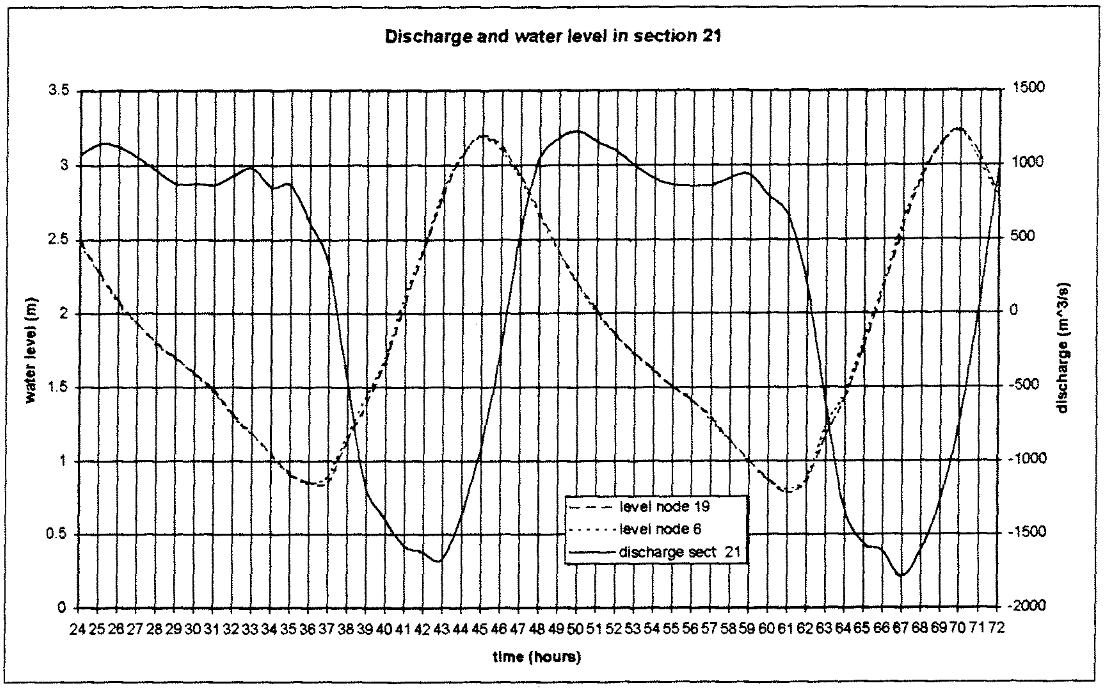

Figure 3—23 Discharge and water levels in section 21

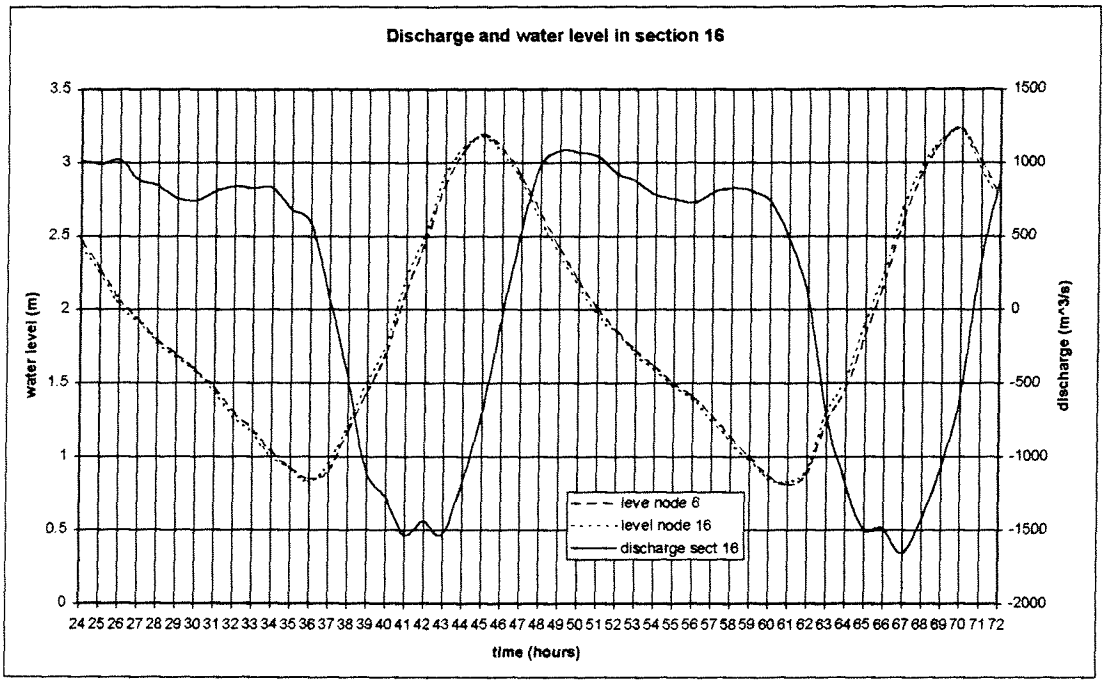

Figure 3—24 Discharge and water levels in section 16.

3.6 Results of the calibration

phase difference tidal boundary condition

From [6] the tide tables are available for the mouth of the Musi river estuary. The levels which are given here were used in the Musi River Model (see paragraph 3.3.4.3). However, in the course of this study it was found out that there is a difference between the coordinates given by the tide tables valid for the 'Sungai Musi' (Musi river) and those of the actual place of Musi river mouth (see Table 3-3).

	Longitude (East)	Latitude (South)
Tide tables	104°90	2°20'
Musi river mouth	104°55'	2°15'

Table 3-3 Geographical coordinates, tide tables - Musi river mouth.

If levels deducted from the tide tables are used as a level boundary condition in DUFLOW it is obvious that there will be a phase difference between the measured levels in the Musi river and the calculated levels in the DUFLOW model.

The difference in the geographical coordinates expressed in kilometers equals a distance of X = 65 km. With an assumed average coastal depth of h = 20 m the tidal propagation speed, c, will be:

$$c = \sqrt{gh} = \sqrt{9.8 \times 20} = 14 \text{ [m/s]}$$

The phase difference between the measurements and the simulated data should be in the order of

$$\phi = \frac{X}{c} \times \frac{1}{3600} = 1.3 \text{ [hours]}$$

which can be seen in the figures in paragraph 3.4.

Musi river a tidal basin

In Figure 3—25 the water velocities in different stages at Sungai Lais are compared and in Figure 3—26 the water discharges in different stages at Sungai Lais are compared. It can be seen that the tidal influence is large on the water movement of the Musi river.

The velocities during a tidal cycle is strongly influenced by the tidal range of the spring tide or neap tide. The influence of the upstream discharge is minor. When comparing the extreme velocities during spring tide (or neap tide) these velocities increases with about 15%.

Even in the neap tide period the flow direction reverses, as a result of the tidal influence.

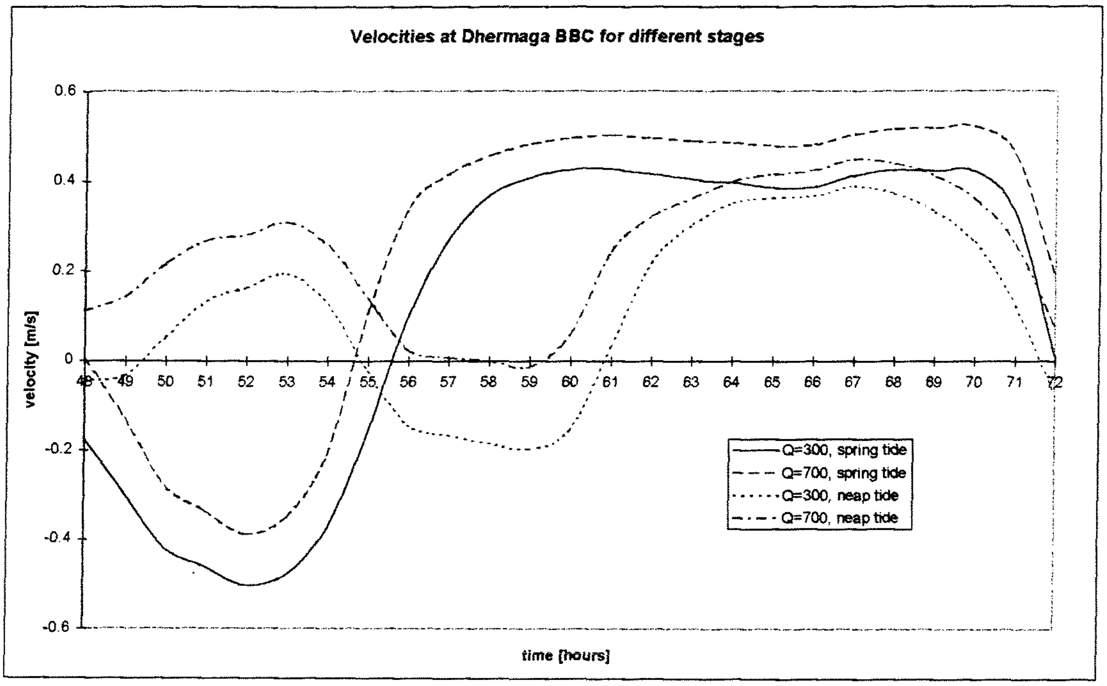

Figure 3—25 Water velocities at Sungai Lais for different stages from DUFLOW.

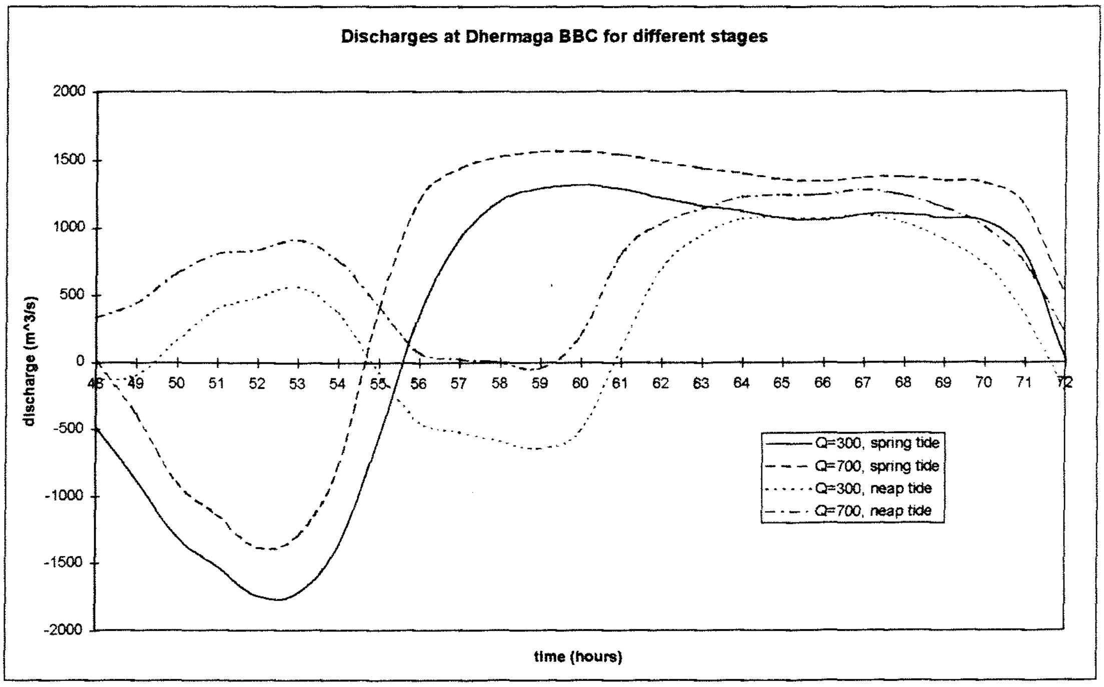

Figure 3—26 Water discharges at Sungai Lais for different stages from DUFLOW

The relation between the upstream discharge and the maximal tidal discharge is $Q_o : |Q_{tida,max}| = 1 : 5$ during spring tide and 1 : 4 during neap tide and the upstream discharge has only little influence.

In Figure 2—3 it can also be seen that the water level range in Palembang (during spring tide in dry season) is still 90% of the tidal range at sea.

Altogether it can be concluded that the Musi river with its strong tidal behavior is more a tidal basin, where the upstream discharges have little influence.

3.7 Closure of the right channel around P. Payung

As mentioned in the problem analysis in paragraph 1.2 on of the alternatives is the closer of one channel around the island P. Payung in the river mouth. The two channel sections (see Figure 3—6) around the island P. Payung in the river mouth are defined as follows:

- left-hand channel = section 8
- right-hand channel = section 11

The left-hand channel is currently used by the larger vessel who sail the Musi river, so the right-hand channel is closed in the simulation. To simulate the effects on the water movement of the left-hand channel three graphs have been made. In the three graphs values can be seen before and after the closure:

- the water levels (see Figure 3—27),
- the water velocities (see Figure 3—28),
- the water discharges (see Figure 3—29).

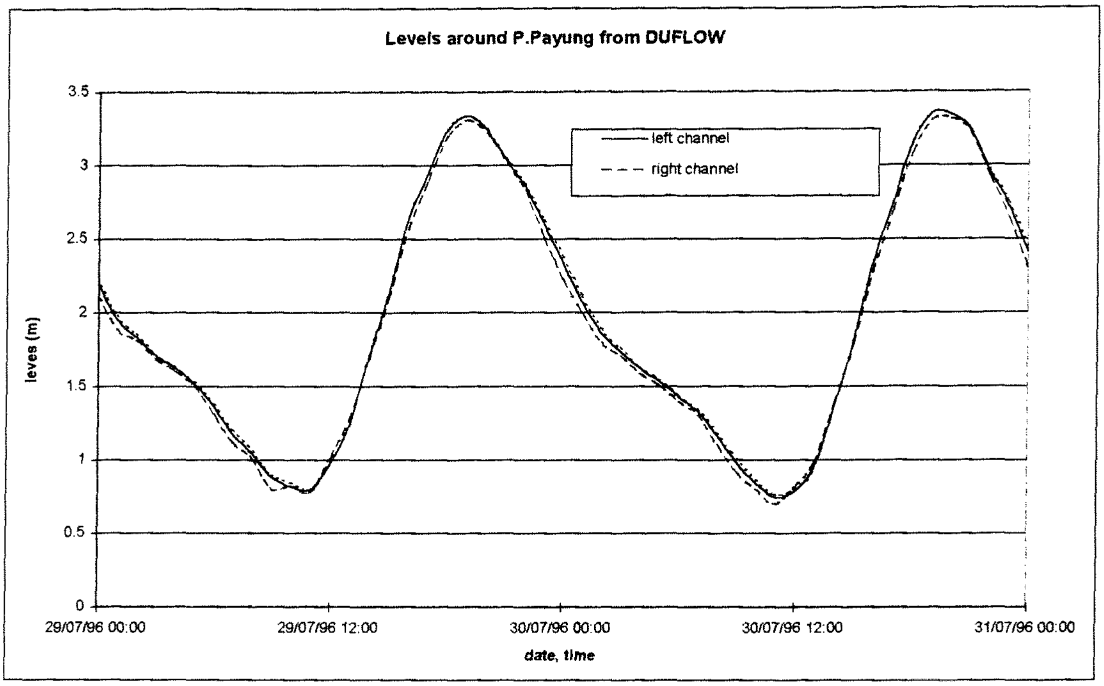

Figure 3—27 Water levels around P. Payung from DUFLOW.

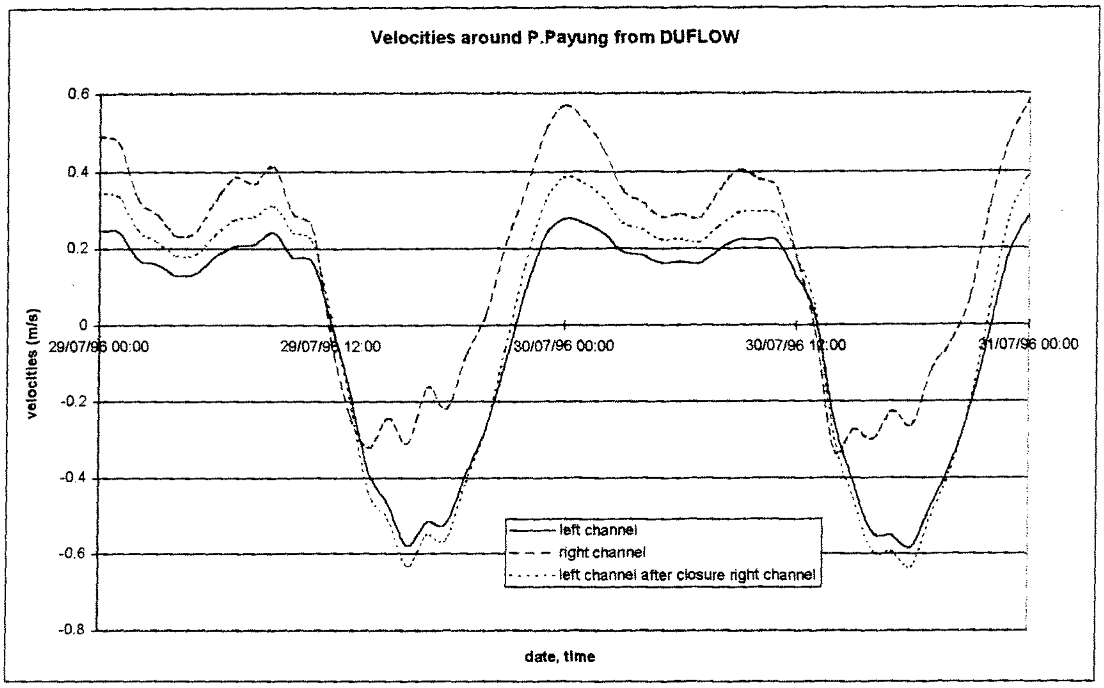

Figure 3—28 Water velocities around P. Payung from DUFLOW.

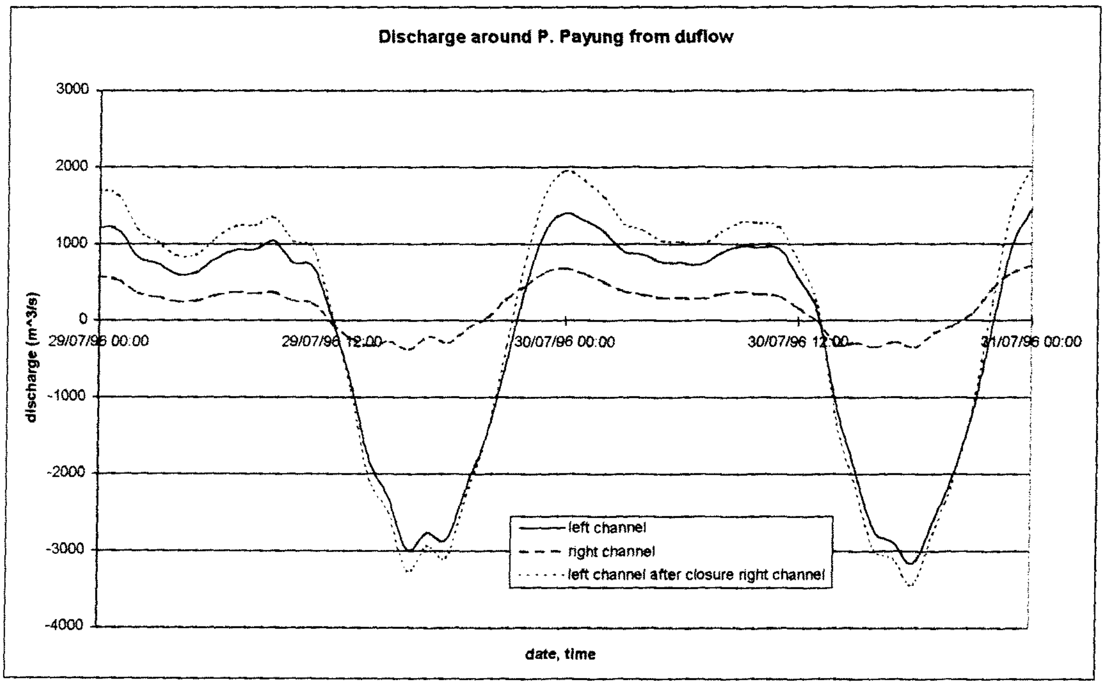

Figure 3—29 Water discharge around P. Payung from DUFLOW.

From the graphs it can be seen that for the left-hand channel:

- the water level hardly changes,

- the maximum water velocity increases from V_{max}=0.28 to 0.36 m/s, which will have some effect on the local sediment transport,
- the discharge increase only for the ebb flow (the discharge of the right-hand channel before closure is relatively small).

The following arguments are also taken into account:

- the average amount which has been dredged in recent years (9 % of total, see Table 2-2) around P. Payung,
- relative high costs of a closure of the right-hand channel.

Based on these arguments and the above presented DUFLOW results it is decided not to go further into *alternative 2*.

3.8 Conclusions

The following conclusions can be drawn from the DUFLOW-modelling of the Musi river:

- The output of the model seems to be sufficiently accurate for this study.
- A Chézy-coefficient of 50 $m^{1/2}/s$ is a reasonable estimation.
- The use of rectangular cross-section profiles in DUFLOW suffices to reproduce the tidal range and phase measurements, but to reproduce the velocity measurements it is preferable to use the cross-section data from the measurements.
- The upstream fresh water discharge amounts to 300 m^3/s during the dry season and 700 m^3/s during the wet season, but does not have much influence on the water levels. The water level range in Palembang is still 90 % of the tidal range at sea. From this can be concluded that the Musi river is more like a tidal basin than a river.
- The model can be used for further studies of the Musi river, but it has to be kept in mind that near Pulau Payung the tidal range is underestimated. This is important since this is one of the shipping traffic bottlenecks in the Musi river, which will be taken into account in the vessel traffic study. The differences at Pulau Payung can be caused by the difference in locations between the actual downstream boundary condition and that of the model, and the water density difference. DUFLOW uses fresh water while in reality salt water is present.
- For further improvement of the model it is recommended to collect additional data, especially on river discharges and bottom slopes. They should not only be collected at certain intervals, but continuously over a number of years.
  These data can be used to improve the Musi River Model for future studies of the Musi river.

4. Siltation in the Musi river

4.1 Introduction

For the calculation of the yearly amount of sediments to be dredged DUFLOW will be used to generate the local flow conditions, i.e. discharge, Q, the flow velocity, V, and the levels, L.

Two principles (see paragraph 2.3) play a role in the siltation in the Musi river. The first is, the transport and settlement of the bed material load which will be treated in paragraph 4.2; the second is the transport and settlement of the wash load, due to flocculation, which will be dealt with in paragraph 4.3.

4.2 Transport and settlement of bed material load.

Bed material load refers to the sediment which is picked up from the river bed and carried either along the bed (bed load) or in suspension (paragraph 2.3). In rivers the settlement of these sediments generally occurs at the transition from a river stretch with a higher sediment transport capacity to a part of the river with less the transport capacity (see Figure 4—1). Siltation (ΔS) will occur in a river section where the outflow (S2) of sediments is less than the inflow (S1) of sediments, as a result of a decrease in transport capacity:

$$\Delta S = S_1 - S_2 \tag{4-1}$$

The transport capacity has a strong relation with the flow velocity as can be seen in equation (4-7). As a result of a lower flow velocity, due to an increase of the river cross-section, the coarser sediments will settle.

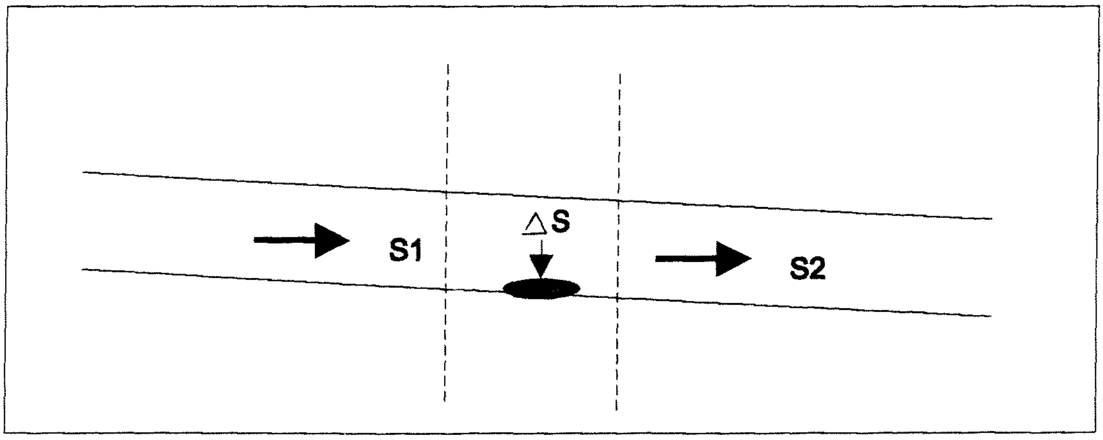

Figure 4—1 Siltation as a result of differences in transport capacity.

With transport formulas like 'Meyer-Peter and Müller' (1945) or 'Englund and Hansen' (1967) the transport capacity of the flow can be calculated. They are valid for a more or less constant average flow velocity.

On the Musi river, however, as a result of the strong tidal influence (see Figure 2—3) the flow reverses and varies daily between a positive (ebb flow) value and a negative (flood flow) value. This means there is no 'constant' flow velocity. In this case the velocity is oscillating (see Figure 2—3). The siltation in the Musi river is assumed to take place according to the following principle (see Figure 4—2):

Where there is no dredging in the Musi river (A1) an equilibrium state is assumed, i.e. there is a constant transport capacity so there is no siltation. At some point a shoal develops and the river has to be dredged every year (B1). The tendency to develop a shoal is caused by:

- increase in width (see Figure 4—2A,B),
- main stream river crossings,
- bifurcation and confluence.

At the Musi river the shoals are dredged and the trenches of the dredged channels fills up from two directions (see Figure 4—2, C), i.e. during the ebb period (equation (4-2)) and during the flood period (equation (4-3)).

$$\Delta S_1 = S_1 - S_1^* \tag{4-2}$$

$$\Delta S_2 = S_2 - S_2^* \tag{4-3}$$

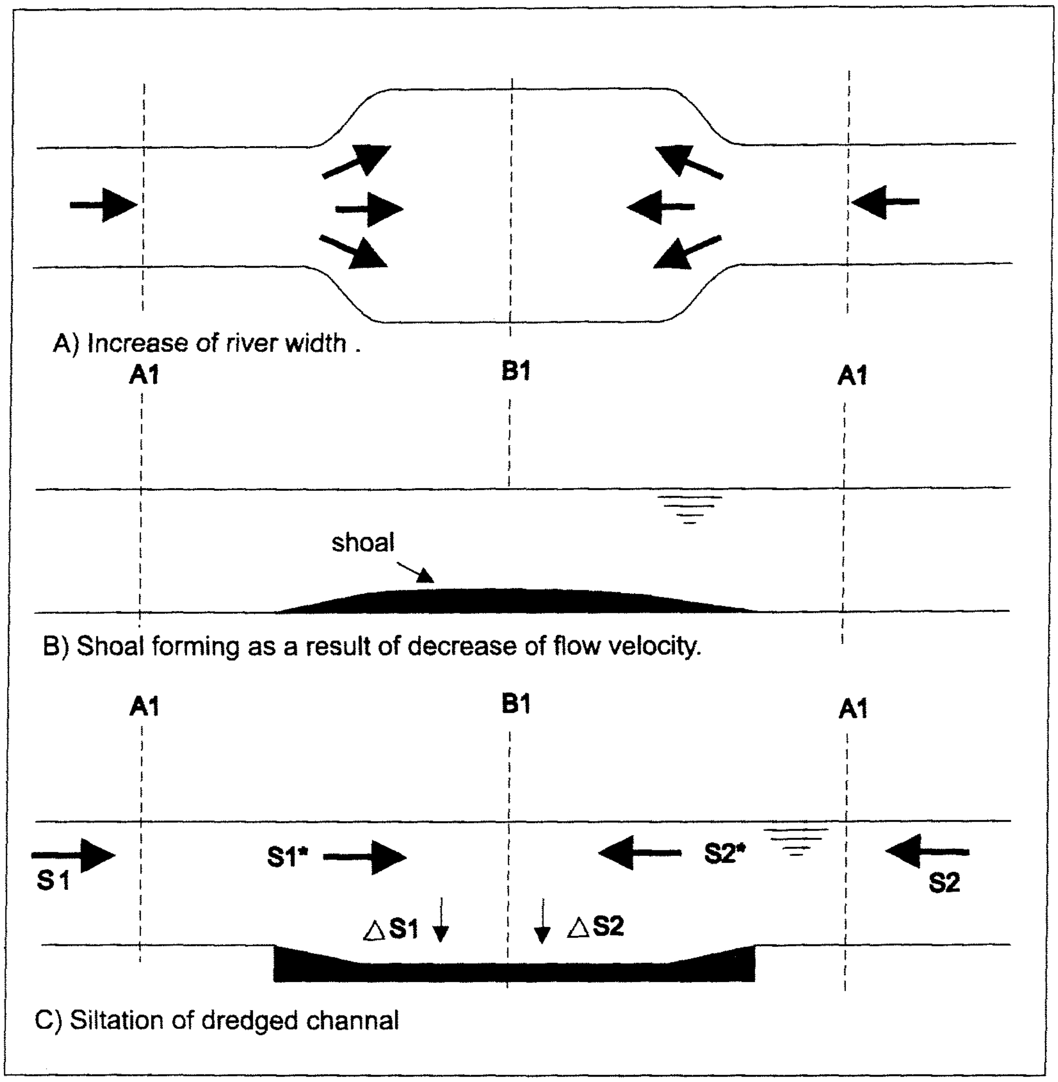

Figure 4—2 Siltation during one tidal cycle in a longitudinal cross section.

To calculate dredging volumes for the different maintenance depths, the following have to be determined:

- present dredging amount
- additional siltation, due to a decrease in transport capacity
- DUFLOW calculations

The present dredging amounts for the different locations can be read from the dredging statistics and are the point of departure for a calculation of the siltation for the different maintenance depths. The dredged volumes are given in Table 4-2. The calculation is based on the fact that the figures given in the dredging history include,

- fresh deposited sediments,
- resiltated material that has been dredged and dumped in resent years (a part of the dredged soil is dredged every year).

The present maintenance level is LWS -6.5 m, which is due to the yearly dredging, naturally not an equilibrium depth.

The yearly amount of additional siltation is then calculated:

$$\Delta S_y = \Delta S_1 + \Delta S_2 \tag{4-4}$$

as the difference in transport capacity when the maintenance level increases from LWS -6.5 to LWS -7.0. The results are given in Table 4-2.

4.2.1 Calculation of yearly amount of material to be dredged.

This method has been developed because the usual sediment transport calculation, as applied to a river with a constant discharge, velocity and water level, is not possible in this case, as explained in paragraph 4.2.

First a few remarks:

- The following method is only valid for that part of the river where no flocculation occurs (see paragraph 4.3), i.e. between the Port of Palembang and Pulau Ayam.
- The DUFLOW model described in Chapter 3 is used to simulate the input values for the sediment calculation.
- As an example the location of Selat Jaran is discussed.

Calculation

Step 1

There are four different situations for which *yearly transport capacities* are calculated, of which an average has to be estimated (see equation (4-5)):

1. Dry season, neap tide, V_A.
2. Dry season, spring tide, V_B.
3. Wet season, neap tide, V_C.
4. Wet season, spring tide, V_D.

The following is done to make an estimation of the actual yearly transport:

The dry season and wet season each lasts 6 months, so the total yearly transport is the average of these two periods. The transport of each period is calculated as a weighted average of the spring tide transport and the neap tide transport.

The influence of the volume transported during spring tide is relative higher than the volume transported during neap tide, which holds for both the wet season as well as the dry season. As can be seen in Figure 2—2 the spring period lasts relatively longer than the neap period. For this reason the a multiplication factor of 1.3 is introduced, which is an estimation. The formula then looks as follows:

$$V_{total,yearly} = \frac{\left[1.3 \times \left(\frac{V_B + V_D}{2}\right) + \left(\frac{V_A + V_C}{2}\right)\right]}{2} \qquad (4\text{-}5)$$

Step 2

For each of these 4 situations,

- DUFLOW runs[1] were made for
    - discharge, Q,
    - flow velocity, V and
    - water level, L.
- the transport capacity was calculated for four different maintenance depths:
    - LWS -6.5 m
    - LWS -7.0 m
    - LWS -7.5 m
    - LWS -8.5 m

Calculation of transport capacity for 1 tidal cycle.

An example of a detailed calculation for S. Jaran is given at one depth (LWS -8,5).

[1] It is checked whether a decrease of the DUFLOW output interval time (10 minutes) gives a more accurate result, but the one hour output interval of DUFLOW seems to be sufficiently accurate for the calculation. The results improved less than 10 %.

One tidal cycle takes 24 hours and is divided into 24 parts of one hour (equal as the DUFLOW output interval).

The following is a calculation of the transport capacity with Englund & Hansen for one tidal cycle:

$$m = \frac{0.084}{g^{\frac{1}{2}} C^3 \Delta^2 D_{50}} \tag{4-6}$$

$$s = mu^5 \qquad \left[\frac{m^2}{s}\right] \tag{4-7}$$

$$S = Bs \qquad \left[\frac{m^3}{s}\right] \tag{4-8}$$

In the last column of Table 4-1 the hourly transport is calculated:

$$V_t \,/\, hour = S{*}3600 \qquad \left[\frac{m^3}{hour}\right] \tag{4-9}$$

The calculation is done with a spread sheet and looks as follows:

time	discharge	water level	depth	formula	velocity	m	u^5	s	S	V/hour
1	-718	0.90	10.0	E&H	-0.16	0.001577	-9.82E-05	-1.5E-07	-6.58E-05	-0.236933905
2	-1467	1.19	10.3	E&H	-0.31	0.001577	-3.06E-03	-4.8E-06	-2.05E-03	-7.385707286
3	-1767	1.59	10.7	E&H	-0.37	0.001577	-6.50E-03	-1E-05	-4.35E-03	-15.67620159
4	-2015	1.96	11.0	E&H	-0.40	0.001577	-1.07E-02	-1.7E-05	-7.16E-03	-25.7825693
5	-2831	2.41	11.5	E&H	-0.55	0.001577	-4.85E-02	-7.6E-05	-3.25E-02	-116.9503262
6	-2688	2.80	11.9	E&H	-0.50	0.001577	-3.20E-02	-5E-05	-2.14E-02	-77.19359548
7	-2947	3.13	12.2	E&H	-0.54	0.001577	-4.44E-02	-7E-05	-2.98E-02	-107.2107888
8	-2176	3.35	12.4	E&H	-0.39	0.001577	-8.96E-03	-1.4E-05	-6.01E-03	-21.62754737
9	-606	3.31	12.4	E&H	-0.11	0.001577	-1.53E-05	-2.4E-08	-1.03E-05	-0.036942551
10	840	3.00	12.1	E&H	0.15	0.001577	8.79E-05	1.39E-07	5.89E-05	0.212051167
11	775	2.67	11.8	E&H	0.15	0.001577	6.71E-05	1.06E-07	4.50E-05	0.161941957
12	1122	2.41	11.5	E&H	0.22	0.001577	4.72E-04	7.45E-07	3.16E-04	1.139160466
13	1693	2.21	11.3	E&H	0.33	0.001577	4.03E-03	6.35E-06	2.70E-03	9.715051857
14	1663	2.03	11.1	E&H	0.33	0.001577	3.98E-03	6.27E-06	2.67E-03	9.59484646
15	1650	1.85	10.9	E&H	0.33	0.001577	4.12E-03	6.5E-06	2.76E-03	9.949789642
16	1382	1.73	10.8	E&H	0.28	0.001577	1.79E-03	2.82E-06	1.20E-03	4.319732151
17	1361	1.62	10.7	E&H	0.28	0.001577	1.74E-03	2.74E-06	1.16E-03	4.192745968
18	1312	1.52	10.6	E&H	0.27	0.001577	1.51E-03	2.38E-06	1.01E-03	3.640744768
19	1269	1.43	6.5	E&H	0.41	0.001577	1.23E-02	1.94E-05	8.24E-03	29.65248823
20	1216	1.35	10.4	E&H	0.26	0.001577	1.11E-03	1.75E-06	7.45E-04	2.682405268
21	1524	1.23	10.3	E&H	0.33	0.001577	3.64E-03	5.74E-06	2.44E-03	8.780337388
22	1620	1.08	10.2	E&H	0.35	0.001577	5.30E-03	8.35E-06	3.55E-03	12.77672203
23	1460	0.95	10.0	E&H	0.32	0.001577	3.34E-03	5.27E-06	2.24E-03	8.058668412
24	1542	0.81	9.9	E&H	0.34	0.001577	4.69E-03	7.39E-06	3.14E-03	11.31402054
									total	488

Table 4-1 Calculation of hourly transport capacity at S. Jaran (dry season spring tide).

With:

time = DUFLOW output interval in [hour].

discharge = discharge calculated by DUFLOW in [m^3/s].

water level = water level calculated by DUFLOW in [m].

depth = water depth, summation of bottom level (DUFLOW definition) and water level in [m].

formula = determination whether Englund & Hansen or Meyer-Peter & Muller should be used. It turned out that on the criterion of $u^*/w < 0.75$ Englund & Hansen had to be used in all cases.

The fall velocity, w, for the sediment particles is calculated using with the formula for turbulent motion, valid for $1 < d < 100$ μm [20] ,

$$w_s = \frac{(s-1)gD_{50}{}^2}{18\upsilon}$$

where:

D_{50} = sieve diameter

s = specific gravity (= 2.65)

v = kinematic viscosity coefficient (= 1.0e-6)

velocity = velocity calculated by DUFLOW in [m/s].

Calculation of total transport capacity in 1 year:

To calculate the transport per day a summation over 1 day is made (see Table 4-1):

$$V_t'' = \sum_{1}^{24} V_t' = 488 \qquad \left[\frac{m^3}{day}\right]$$

To calculate the transport per year, $V_{t''}$ is multiplied by the number of days in a year:

$$V_t = V_B = V_t''*365 = 178.226 \quad \left[\frac{m^3}{year}\right]$$

This is done for the 4 distinguished situations:

V_A = 8,758

V_B = **178.226**

$V_C =$ 10,271

$V_D =$ 178,226

To determine the transport capacity per year a weighted average of the spring tide transport and the neap tide transport is estimated:

$$V_{total,yearly} = \frac{\left[1.3 \times \left(\frac{V_B + V_D}{2}\right) + \left(\frac{V_A + V_C}{2}\right)\right]}{2} = \mathbf{119{,}990} \left[\frac{m^3}{year}\right]$$

The same procedure can be followed for the different depths:

LWS	capacity	difference	S. Jaran
-6.5	172,381		96,336
-7.0	150,581	21,800	118,136
-7.5	139,425	32,957	129,293
-8.5	**119,990**	52,392	148,728

Figure 4—3 Transport capacity at different depths near S. Jaran.

The same calculations, described in the example of S. Jaran, can be performed at the other locations for different maintenance depths. The results are given in Table 4-2:

LWS	S. Lais	A. Kumbang	S. Jaran	S. Upang	P. Ayam	Total
***-6.5**	83,343	24,065	96,336	62,217	52,555	318,515
-7.0	91,713	29,518	118,136	65,173	56,537	361,077
-7.5	99,344	34,493	129,293	69,332	56,030	388,492
-8.5	112,680	42,764	148,728	76,881	58,698	439,751

Table 4-2 Present (-6.5) and calculated dredging amounts on river shoals in m^3/year.*

4.3 Transport and settlement of wash load

Wash load refers to sediment which originates from other areas of the catchment area, is finer than the bed material, and is carried through the river in suspension (see paragraph 2.3). It should be mentioned that the amount dredged near the river mouth and at the outer bar is 85 % of the total dredged amount over the past years. The amount of sediments dredged near the outerbar is settled due to the flocculation and decrease of flow velocity in that area.

Below LWS	P.Payung/outerbar
6.5	2,781,485
7.0	3,538,923
7.5	4,411,508
8.0	4,885,879
8.5	5,360,249

Table 4-3 Volumes at P. Payung (river mouth) and the outerbar in m^3/year.

The volumes of Figure 4—3 are empirical values and estimated as the difference between the resent dredging history and the dredging history of [10] (see Figure 6—6) and the calculated amounts at the river shoals of the previous paragraph.

4.4 Conclusions

- The amount that will have to be dredged on the Musi river upstream of the river mouth is only 15 %. When river works are constructed along the Musi river a reduction in dredging amount of 15 % at most can be achieved.
- For reasons of performing a more thorough analysis on the sedimentation process more specific data is required especially near the river mouth and the outer bar, which will be dealt with in the recommendations in chapter 8.

5. Riverworks in the Musi river

5.1 Introduction

Basically the width of the river is to large to lead to the required depth. Hence, restriction of the width is a possible solution to reduce the mount to be dredged every year. Local width-restriction downstream of the Palembang may be feasible.

The potential improvement of the river by building groynes at the inner bends at certain locations has been suggested by Haskoning [10]

Although only some 13 % (see Table 2-2) of the total amount dredged is in the river reach 60 km downstream of Palembang, the fact that this has to be dredged at various places makes it relatively expensive. Moreover there will be the tendency to dump the soil rather close to the dredged areas (see Figure 6—4). This makes it possible that the sediment will reach the dredged channel again. A distinction has to be made between *open groynes* and *closed groynes* [27].

5.2 Two alternative

The following two alternatives concerning the riverworks will be investigated as mentioned in paragraph 1.2:

- Set 1 of riverworks: groynes at shoals along the Musi river to constrict the flow width and maintaining the rivers depth through dredging.
- Set 2 of riverworks: groynes at shoals along the river + closure of one branch around P. Payung (island in the river mouth) and maintaining the rivers depth through dredging.

5.3 Construction methods

5.3.1 Open groynes

By placing open groynes at he inner bends of a river the cross-section is divided in two parts. In the inner lane the water is passing the groynes. The groynes create resistance to the water. Thus the velocity in this lane is reduced. Consequently the velocity in the outer lane is increased. This leads via erosion to an increase to an increase of the depth in the *outer lane*. The lengths of groynes and the distance between the individual piles of the groynes determine which increase of the depth in the *outer lane* can be reached.

For relative coarse bed-material sedimentation in the *inner lane* can be expected due to the velocity reduction. This will gradually increase the erosion in *the outer lane*.

For the relative fine bed material present in the Musi river (see paragraph 2.2) it is questionable whether open groynes will work this way. This due to the fact that the piles of the groynes create extra turbulence to the flow, increasing the sediment transporting capacity in spite of a reduction of the mean flow velocity.

The key parameter in this respect is the ratio (β):

$$\beta = \frac{\text{fall velocity of sedimet (w)}}{\text{shear velocity } (v_*)}$$

The shear velocity (v_*) can be found from $\frac{v}{v_*} = \frac{C}{\sqrt{g}}$ in which C is the Chézy roughness coefficient.

For the fine sediment present the fall velocity (w) can be found from Stokes' Law (for D < 0.1 mm).

$$w = \frac{1}{18}\frac{\Delta g D^2}{\upsilon}$$

Sediment transport in suspension is present for $\beta < 1$. The smaller β is, the more uniformly is the sediment concentration distributed over a vertical.

The extra turbulence created by the piles makes that the sediment will be distributed more uniformly over the vertical than the β value present indicates. This restricts the possible sedimentation on the *inner lane*. This reduces the deepening at *the outer* lane that can be expected due to the reduction of the flow velocity in the inner lane only.

Consequently without further detailed research on the effect on open groynes on the sediment transporting capacity in the inner lane it is advised not to apply open groynes for the lower Musi river .

5.3.2 Closed groynes

This type of groynes have been and are used as a measure to reduce the width of a river. The remaining channel will get then a larger specific discharge leading to erosion. Thus the depth increases. This is a time-depending process.

For a flow in one direction this leads temporarily to sedimentation downstream of the narrowed reach. Although in a tidal river this sedimentation is basically smaller as test computations by Haskoning (1984) indicate, this sedimentation may locally and temporarily be a hindrance to navigation.

A practical solution to avoid the temporary downstream sedimentation the estimated bed-level of the narrow channel of the river can be dredged.

The design of the closed groynes can be based on the measurements of flow velocities across the river in a typical cross-section of the reach to be narrowed.

It has to be warned, however, that the application of groynes in inner banks is only effective on the long run if the erodibility of the outer bank is restricted. This is important as the banks of the Musi river do not gave artificial bank protection.

During the inspection tour on the Musi river (November 1996) the writer got the impression that the presence of mangroves all along the Musi river as far as P. Payung Island may be a guarantee of resistance against bank erosion. It is however not possible to draw this conclusion from an inspection at one time.

5.4 Total length of groynes and costs

The groynes can be constructed of local materials such as wood, which need probably some more maintenance but makes it easier to maintain.

The costs are estimated at Dfl. 175,- /m' .The total length of the groynes needed near the shoals is estimated at 13,000 m' [10]. No depreciation is required, cost being accounted for by 12 % yearly interest on the investment and 10 % yearly maintenance costs, which includes the progressive replacement of damaged parts.

5.5 Conclusions

- Because it is not sure whether the river banks covered with mangroves have enough resistance against the bank erosion, it is recommended to compare aerial photographs of the Musi river in order to conclude that the planform of the river bank has been reasonably stable in the past.
- When it is considered to construct groynes it is advisable to initiate the constructions of a groyne system on one of the easier locations to observe and monitor the channel development on order to improve the layout and design of subsequent groyne systems at more complicated locations.

6. Dredging on the Musi river

6.1 Introduction

A specification of the dredging costs depends on the maintenance level (LWS -6.5 m, LWS -7.0, LWS -7.5) and the applied set of riverworks, dredging methods, dumping areas and the possibility of reusing the material for reclaiming purposes. The costs of capital and maintenance dredging for the three different levels will be determined for each of the alternatives.

In paragraph 6.2 a short description of the types of dredging vessels used is given, as well as an option for the future. In paragraph 6.3 a some observations are made about the disposal of dredged materials. In paragraph 6.4 the amount of material to be dredged for various maintenance depths is given. Subsequently, paragraph 6.5 gives the calculation of the dredging costs for the various maintenance depths. Finally, conclusions and recommendations are given in paragraph 6.6.

6.2 Dredging equipment

At present, dredging is performed by two trailing *suction hopper dredgers*, i.e. *Bali* and *Floris*, which are owned by the Government Dredging Company Rukindo. The known specification of the dredging vessels are given in Table 6-1.

	Floris	**Bali**
Hopper capacity [m^3]	2,200	5,000

Table 6-1 Hopper capacity of two trailing suction hopper dredgers.

Because of the scarcity of information about the used vessels is available, the following standard characteristics are assumed from [25] and will be used from now on (see Table 6-2).

	Floris*	Bali*
Hopper volume [m³]	2,400	4,700
Displacement on dredging mark[1] [t]	5,200	9,900
Weight of 'lightship' [t]	1,680	3,125
Standard[2] value [Dfl.]	32,090,000.-	54,370,000.-
Propulsion [kW]	2,200	5,300
P_z power on the ground pumps during suction [kW]	1,150	1,950

Table 6-2 Characteristics of two trailing suction hopper dredgers.

It should be kept in mind that both vessels mentioned above are of a modern type. This means that the hopper can be fully loaded with a soil density of,

$$\rho = \frac{\text{displacement on dredging mark - weight of 'lightship'}}{\text{hopper volume}} \approx 1450 \text{ [kg/m}^3\text{]} \qquad (6\text{-}1)$$

6.2.1 Trailing Suction Hopper Dredgers

Trailing suction hopper dredgers like *Bali* and *Floris* are vessels suited to coastal (around Pulau Payung and at the outerbar) or deep sea navigation, which have the ability to load a hopper contained within its structure by means of a centrifugal pump or pumps whilst the vessel is moving. They have twin screw propulsion and a powerful bow thruster, which provide a high degree of manoeuvrability.

In Figure 6—1 the most important features of trailing suction dredgers are illustrated.

The main advantages of the trailing suction dredger are:

- relative immunity to weather or sea,
- independent operation,

[1] Displacement on dredging mark = weight of 'lightship' + deadweight.

[2] Based on the 1995 index value of 100, [25]

- minimal effect on other shipping,
- the ability to transport dredged material over long distances,
- relatively high rate of production,
- simple, and hence inexpensive, mobilization procedure.

The main disadvantages are:

- inability to dredge stiff cohesive or rock type soils,
- inability to work in very restricted areas,
- sensitivity to concentrations of debris,
- dilution of dredged materials during the loading process.

The trailing suction hopper dredger is normally rated according to its maximum hopper capacity, which typically lies in the range of 750 to 10,000 cubic meters, although in a few cases it is larger [4].

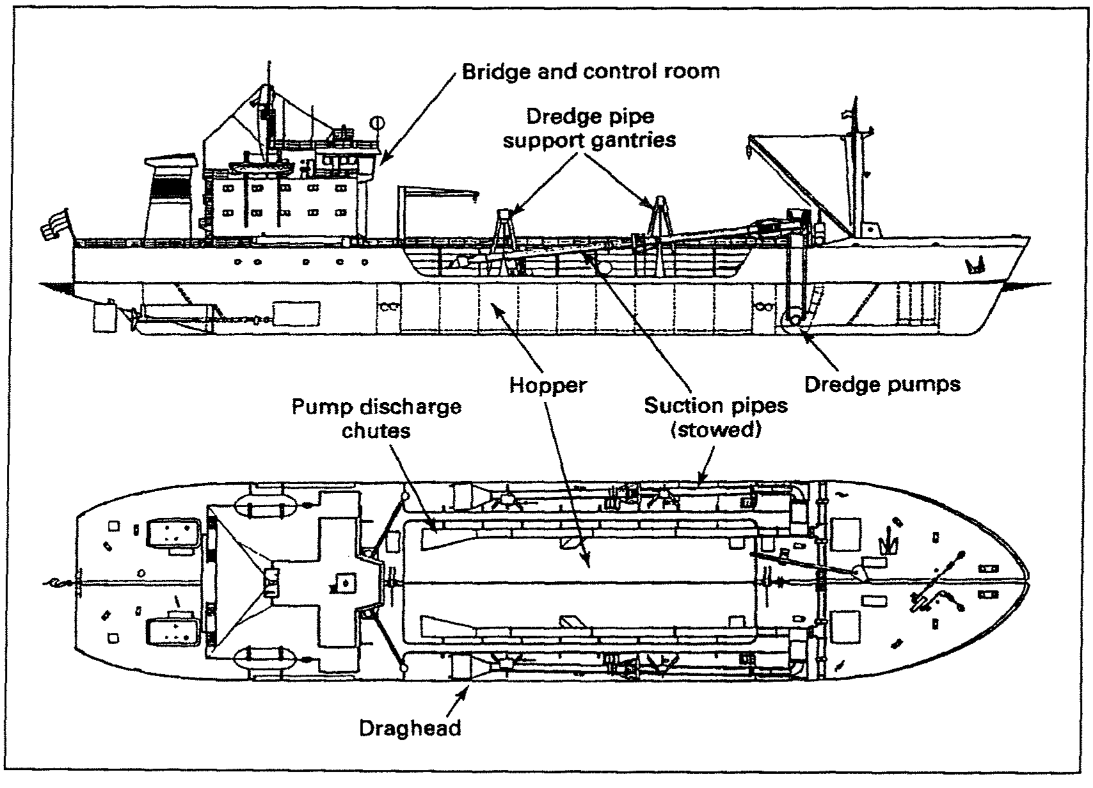

Figure 6—1 Main features of trailing suction hopper dredgers.

Loading takes place while the ship moves slowly ahead (2-4 knots). Unloading is normally done by means of a bottom-discharge arrangement or by pump discharge, in which case unloading is usually done to the shore (see 6.3.2). The maximum depth to which dredging is possible is limited by the length of the suction pipes or by the suction process. This is however not a limiting factor on the Musi river.

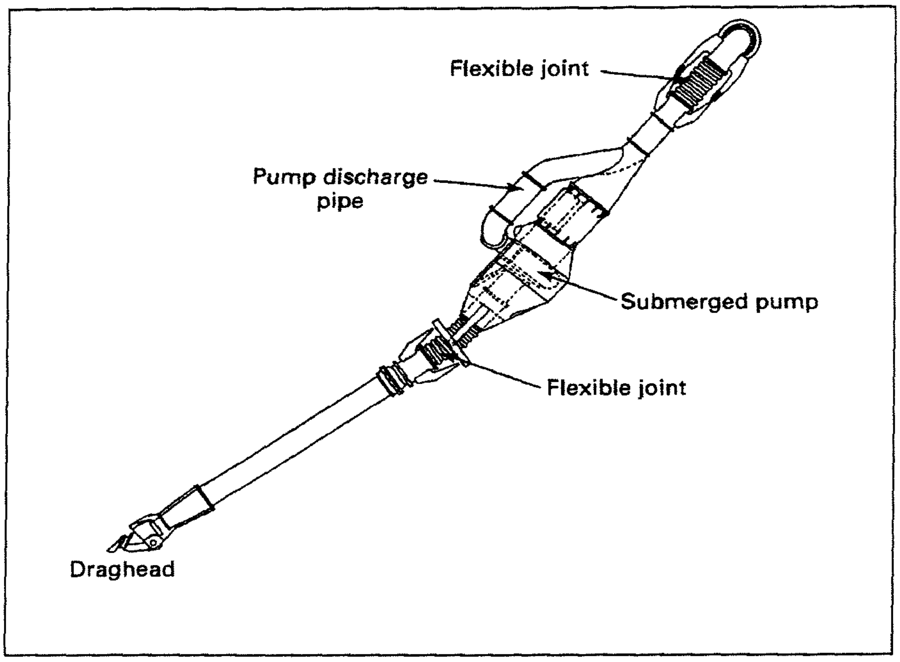

Figure 6—2 Suction pipe of trailing suction hopper dredger, fitted with submerged pump.

Within the hopper, some or all solids in the pumped mixture will settle out of the suspension and the supernatant water is discharged through an overflow. Very fine materials may not readily settle from suspension, which is the case at P. Payung (mouth) and the outerbar. When dredging these types of materials, there will usually not be any significant increase in load achieved by continued pumping after the point of hopper overflow. Most trailing dredgers are designed to carry a full load of fine grained material. They are not generally able to carry a full load of sand or gravel because of the greater density of the material in the hopper and may only be able to load to 80 per cent of hopper capacity in these materials.

6.2.2 Water injection dredge

A dredging method which currently[1] is not (yet) applied, but could bean option for the future is WID (Water Injection Dredgers). Location where water injection could be applied is at the mouth and the outerbar of the Musi river, because of the silty soil.

The principle of water injection systems is that by the injection of water into certain types of sea bed materials, the *in situ* soil density is reduced to the point where it behaves as a liquid and is mobilized by gravity. The method was developed and exploited commercially during the 1980's in the Netherlands. (see Figure 6—3)

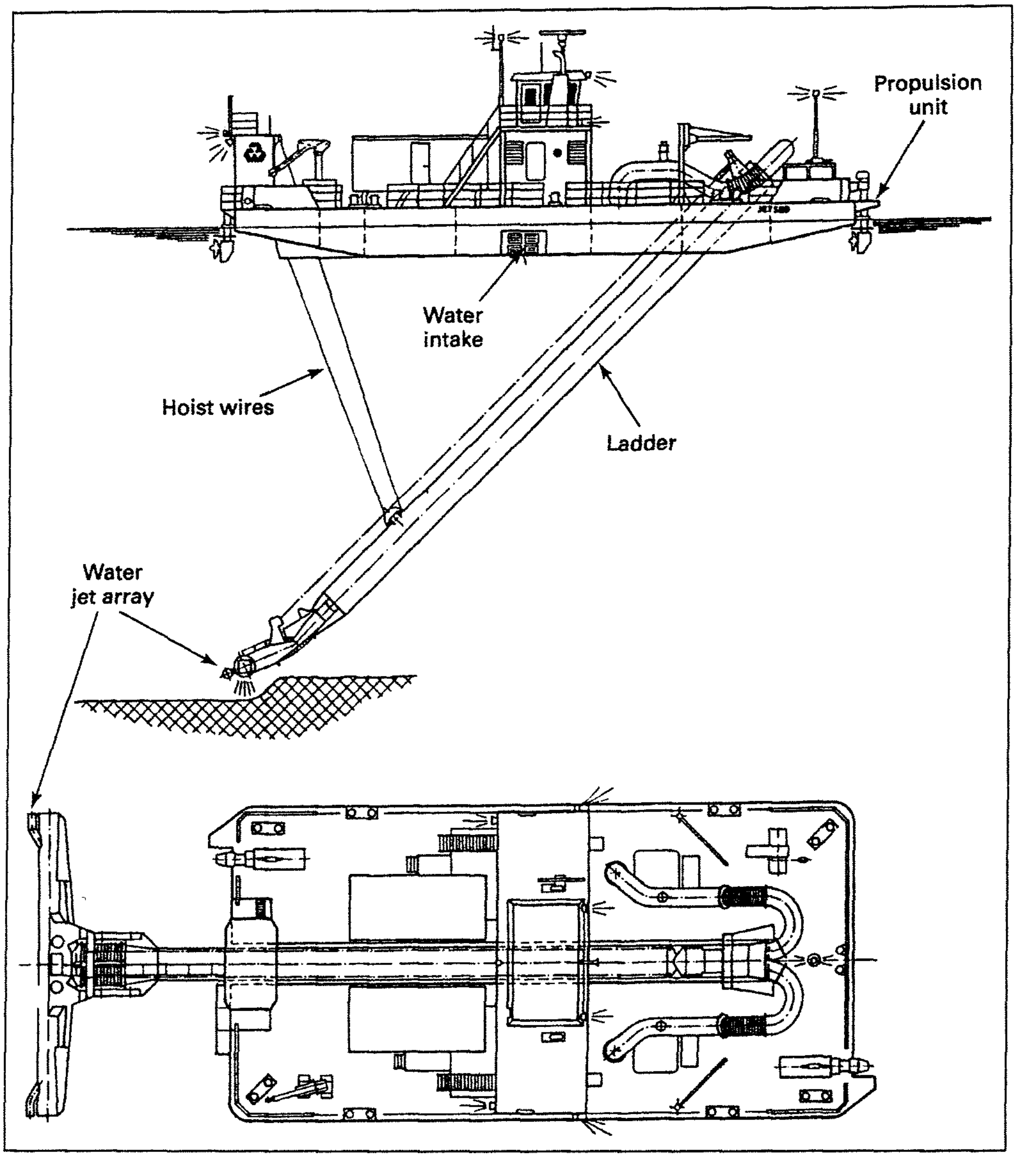

Figure 6—3 A water injection dredging system.

The method is most successful in low strength fine-grained materials, when the fluidised material can flow to a lower level. The objective is to create a thin layer of density currents at the bottom. The grains are carried by these currents to a pre-defined location. The water injection unit is self-propelled. When working, a fixed array of water jet nozzles is lowered to the seabed, water injection is initiated at pre-

[1] 1997

determined pressure and flow rate and the vessel moves slowly ahead creating the fluidised material. If the seabed slopes away from the working area, mass flow of the fluidised material may occur over considerable distances and at very high flow rates. The method could still be used at the Musi river mouth, where the bed is on level or, at certain points, on undulating ground.

6.3 Disposal of dredged material

6.3.1 Dumping areas

The disposal of dredged material is a yearly recurrent problem on the Musi river. The dredging contractor dumps the dredged soils at the nearest location (see Figure 6—4), which is an appropriate place according to them, without any consideration whether the dredged material will stay there or may cause resiltation. This is one of the reasons, apart from the state subsidy and low local wages, why the dredging price per cubic meter is very low (according to Rukindo: Rp. 2550,-/m^3 = +/- $ 1,- US, 1996). No data are available for the ratio of dredged material that will settle to the amount that will resiltate, immediately or after a while, at the river shoals.

Another aspect which plays a role in the river system is the yearly withdrawal of sediments. It is not known what influence this may have on the mangrove at the river banks, which play an important role in maintaining the stability of the river banks.

Some options for dumping the dredged material will be discussed in the next paragraphs.

6.3.2 Alternative locations

The dumping locations given in Figure 6—4 have been used in recent years. It is hard to give any quantitative analysis because of the scarcity of information about the local conditions. The following has to be kept in mind:

- The dumping locations 1, 3 and 4 are near islands in relative shallow parts of the river, i.e. confluences and bifurcations. Local velocity patterns have to be determined to analyze the bottom stability of these locations.
- The dumping locations 2, 5 and 6 in Figure 6—4 are situated in a tributary of the Musi river. When dumping at those locations the local depth will decrease and as

a result the flow velocity and thus the local transport capacity will increase. The resent dumped soil will flow back to the river, because, locally, the river will want to re-establish its equilibrium cross-sectional area (mainly depth).

- The dumping locations 7 and 8 are beyond the outerbar, where the water depth is +/- 20 m. It is possible that the dumped soil will return to the outerbar or the river's mouth. Local conditions such as density currents and tidal currents along these coast will have their influences and they have to be investigated.

Another alternative location is the following: the sand dredged between Palembang and Pulau Ayam (with the exception of the two silt locations) is used for building projects in Palembang. At present[1] Rukindo is running tests concerning the methods for transporting the riversand from dredging vessels to this site.

For the test a 700 m long pipeline has been laid from a dredging vessel in the Musi river to a test location. The future construction area in Palembang is situated south east of the big Palembang bridge and has a length of 2,7 km.

The profit is about Rp. 400 per m^3. Although in this way it is possible to make money out of it, this will not be taken into account in the cost calculation, because no information is available on the future amounts or needs.

[1] December 1996

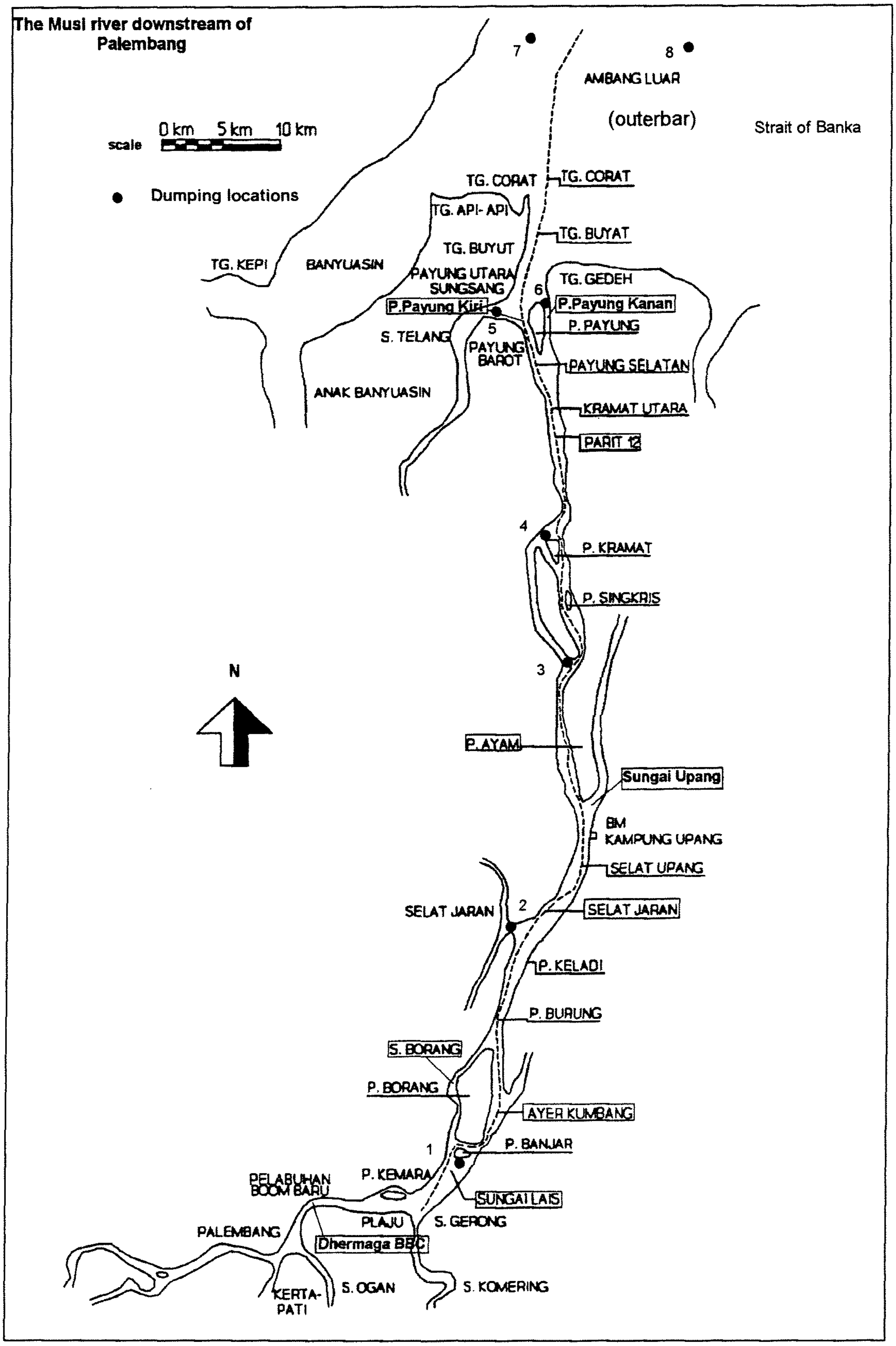

Figure 6—4 Dumping locations along the Musi river in recent years.

6.3.3 Reuse for land reclamation

Another possibility to use the dredged material is reuse for land reclamation projects along the Musi river. Two reclamation projects are under consideration for the future by Pelabuhan II, i.e. the port authorities.

Firstly, there are plans for the near future to expand the port activities of the Boom Baru terminal to Sungai Lais. In order to build the necessary infrastructure new land reclamation is needed.

Secondly, in the future the port of Palembang, i.e. the State Oil & Gas Company Pertamina, the coal terminal and the Boom Baru terminal, wants to expand near Api Api at the mouth of the Musi river, to aid future development.

6.4 Calculation of dredging volumes

6.4.1 Channel width

The current minimum breadth of the navigation channel from the sea to Palembang is maintained at approximately 150 m all along the river [7] [17].

Draught [m]	6.0
Breadth (m)	30
Length Over All [m]	160
DWT [tons]	20,000

Table 6-3 Main characteristics of a representative vessel of Pertamina.

The largest vessels that currently sail to Palembang have a draught of 6 m and a breadth of 30 m. For a two lane channel a width of 150 m and a depth of approximately 6.6 m are recommended based on [1]. This means that the navigation channel is for the most part a two lane channel.

6.4.2 Capital dredging

The resulting volumes that will have to be dredged at different parts of the river are given in Figure 6—4. The calculation of the capital dredging amounts can be found in the Appendix D.

	River shoals	P.Payung/outerbar	Total
Below LWS	**Volume**	**Volume**	**Volume**
6.5	28,249	4,684,321	4,712,570
7.0	670,060	5,993,245	6,663,305
7.5	2,937,407	8,564,019	11,501,425

Table 6-4 Total capital dredging volumes for different parts of the river in m^3/year.

In Figure 6—5 the calculated volumes are compared with the volumes given by [10].

- The volume according to a previous feasibility study start at the depth of LWS - 7,5 m, because at that time[1] the maintenance level was LWS - 7,0 m.
- For the calculation of the capital dredging volume the navigation map [7] was used. It can be seen (in Table 6-4) that at LWS -6.5 m there is little capital dredging needed on the river itself, but near P. Payung (river mouth) and near the outerbar more capital dredging is required.

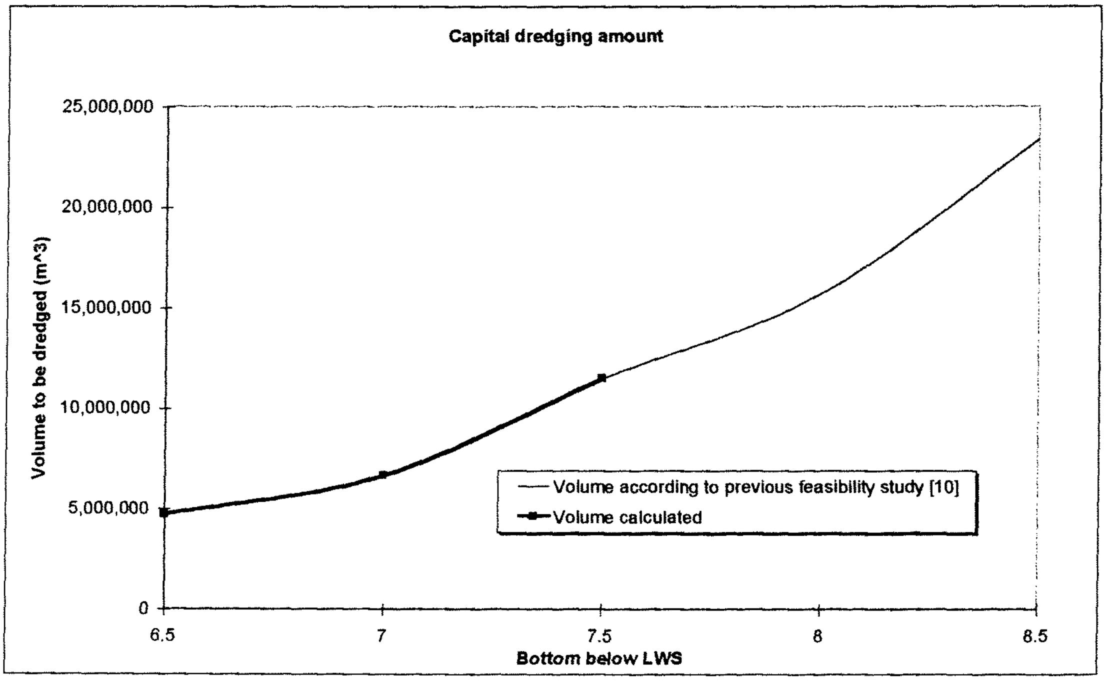

Figure 6—5 Capital dredging volumes

6.4.3 Maintenance dredging

The resulting volumes that will have to be dredged at different parts of the river are given in Table 6-5. They are determined in Chapter 4.

	River shoals	P.Payung/outerbar	total
Below LWS	Volume	Volume	Volume
6.5	318,515	2,781,485	3,100,000
7.0	361,077	3,538,923	3,900,000
7.5	388,492	4,411,508	4,800,000
8.0	414,122	4,885,879	5,300,000
8.5	439,751	5,360,249	5,800,000

Table 6-5 Total maintenance dredging volumes for different parts of the river in m^3/year.

The port authorities of the Port of Palembang, Pelabuhan Indonesia II, say that the maintenance level along the Musi river is LWS - 6.5 m. In Figure 6—6 the maintenance dredging volumes given in [10] are compared with the volumes dredged in recent years. It can be seen that it is more plausible that the volume dredged in recent year matches with a maintenance depth of LWS - 6.0 m.

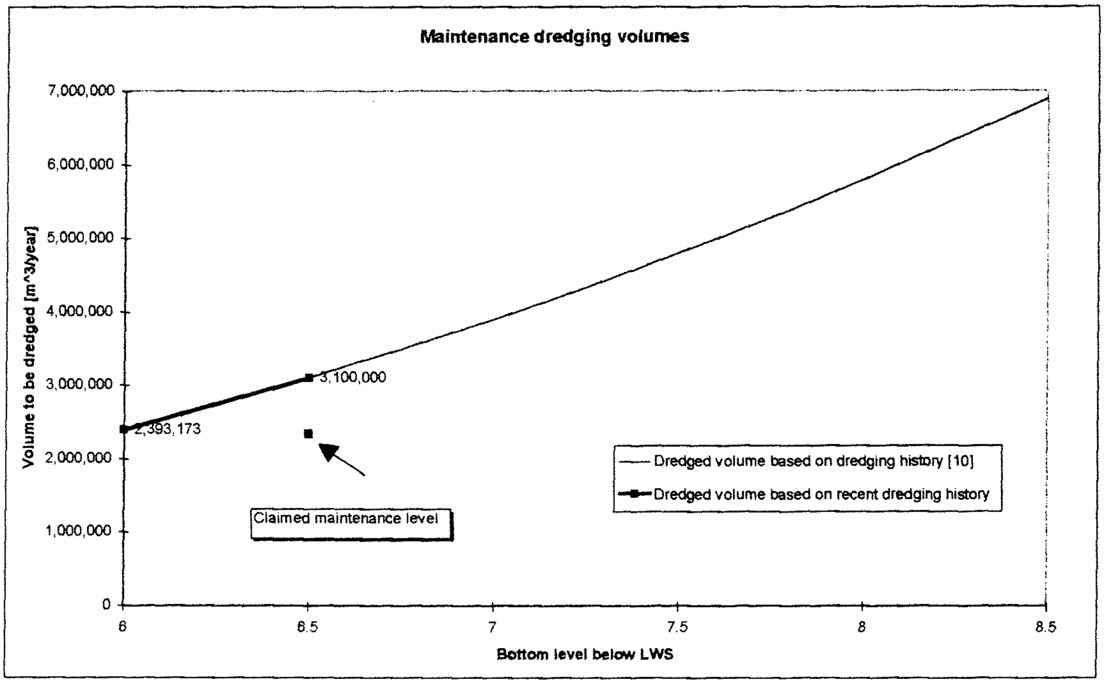

Figure 6—6 History of dredged volumes on the Musi river.

6.5 Calculation of the dredging costs

In this paragraph a cost calculation is made for maintenance levels of the Musi river of LWS -6.5, -7.0 and -7.5. On the basis of the available data and the sediment

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[1] 1984

transport calculations in chapter 4, the sediment volume has been determined at the:

- outerbar,
- P. Payung
- river shoals

Paragraph 6.5.1 deals with the operational costs involved in the process. In paragraph 6.5.2 a calculation method is given to estimate the output of the dredging vessels (detailed calculations can be found in the Appendix D). The determination of the capital dredging costs and the maintenance dredging costs are given in paragraph 6.5.3 and paragraph 6.5.4. Finally the total costs are given in paragraph 6.5.5.

6.5.1 Operational costs

With the average output determined in the previous paragraph we can now calculate the yearly costs. The following price levels are based on information obtained from Mr. G.L.M. van der Schrieck[1].

crew costs

For a trailing suction hopper dredger the following numbers and prices are assumed,

	Floris*		**Bali***	
	Expatriates[2]	Local	Expatriates	Local
number	7	7	15	15
cost [Dfl./month]	4,500	1,125	4,500	1,125

Table 6-6 Crew numbers and costs.

fuel and lubricant costs

Fuel and lubricant costs are based on a consumption of 0.20 liter / kW hour and a price of 0.50 Dfl. / liter.

[1] Lecture Dredging Technology TUD.

[2] Technical specialists.

Table 6-8 summarizes the operational cost of the two dredging vessels.

6.5.2 Estimation of output

The following parameters have been taken to estimate the output.

The maximum hopper capacity is H:

	Floris*	Bali*
H in [m_3]	2400	4700

Table 6-9 Maximum hopper capacity.

The distance, g, to the disposal site and the length, l, of the dredging area are average estimated values, based on the dredging history.

A detailed dredging cost calculation can be found in the Appendix D, where the following variables are used,

variable	value	unit	description
g	10	[km]	the distance to the disposal site.
l	2	[km]	the length of the dredging area.
V_g	17	[knots]	the fully laden sailing speed of the dredger.
t_d	4	min	the time taken to dump the dredged material
t_t	5	min	the time taken to turn the dredger at each end of the dredging area.

Table 6-10 Variables used in dredging cost calculation.

The soil types to be dredged are as follows,

	River shoals	P. Payung and Outerbar
Type[1]	fine to medium sand	silt, very soft clay
D_{50} [μm]	60 - 300	6 - 60

Table 6-11 Soil type to be dredged.

[1] Based on 'Particle size classification' of British Standards and M.I.T. [16]

The term 'output' will be used and defined as the *in situ* quantity of material dredged in a given period of time. The output will be calculated and qualified as follows:

Hourly output = average quantity dredged in a working hour.

First the **maximal potential output,** P_{max}, will be determined. This output is achieved in the productive working time. It is a theoretical figure which cannot be achieved in sustained operations and represents the average hourly output in ideal circumstances with 100 per cent efficient crew and machinery in the given site and operating conditions.

In practice, there are no ideal conditions. Therefore, to estimate the **final output**, P, to obtain program periods and budget cost estimates, reduction factors to P_{max} have to be applied.

Delay factor, f_d

Delays due to bad weather and interruptions due to passing maritime traffic can be combined to form a delay factor. It is necessary to express the time which will not be lost due to traffic delays as a fraction of the working time available. This figure should be multiplied by the fraction of time during which weather, and therefore sea conditions, are suitable for working. Because the weather conditions are good and no problems are to be expected (hoppers) the delay factor will be of no influence,

$$f_d = f_t \times f_w = 1 \times 1 = 1 \tag{6-2}$$

where:

$$f_t = \frac{\text{total working time available - time lost due to traffic during working hours}}{\text{total working time available}} = 1$$

and: (6-3)

$$f_w = \frac{\text{total of days (hours) when weather is suitable for working}}{\text{total number of days (hours)}} = 1 \tag{6-4}$$

Operational factor, f_o

Neither crew nor management in a dredging organization can be a 100 per cent efficient. An operational reduction factor is thus necessary to take inefficiencies into account. An average value valid for 'good climate' [4] is

$f_o = 0.75$

Mechanical breakdown factor, f_b

Assuming that after 20 years a dredger will be completely overhauled, the following reduction factor is applied: for the first five years there is no reduction, while for every subsequent year a one per cent reduction holds, which means that after 20 years the mechanical breakdown factor, f_b, = 0.85.

Productive unit

The productive unit for a trailer dredger is the hopper capacity, H.

The productive unit has to be modified by the bulking factor, B, which takes account of the bulking of the dredged material, which in this case is the ratio of the volume of the hopper to the in situ volume. The modified productive unit thus becomes:

$$U_m = \frac{H}{B}$$

and is the total *in situ* volume of dredged material which, theoretically, can be contained in a full hopper (see Appendix D for values of B).

The cycle factor.

The dredging cycle of a trailing suction hopper dredger consists primarily of four components (see Figure 6—7):

- loading (dredging)
- turning the dredger at the end of each run across the dredging area
- sailing to and from the disposal ground
- discharging the dredged material

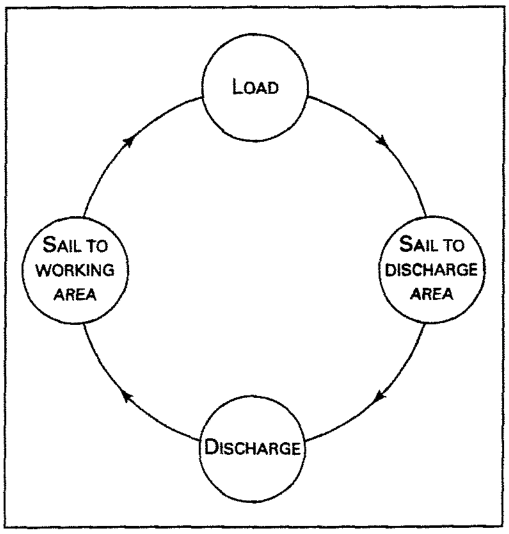

Figure 6—7 Production cycle for the trailing suction hopper dredgers.

Loading

Loading time, dependent on soil type, overflow losses over the hopper weir and attainable concentrations in the suction pipe due to depth of dredging, obstructions and so on. Loading time[1], t_l, against proportion of filled hopper, f_e, is given in Figure 6—8 [4]. The graph sow the lading in terms of loading time, t_l, against proportion filled, f_e.

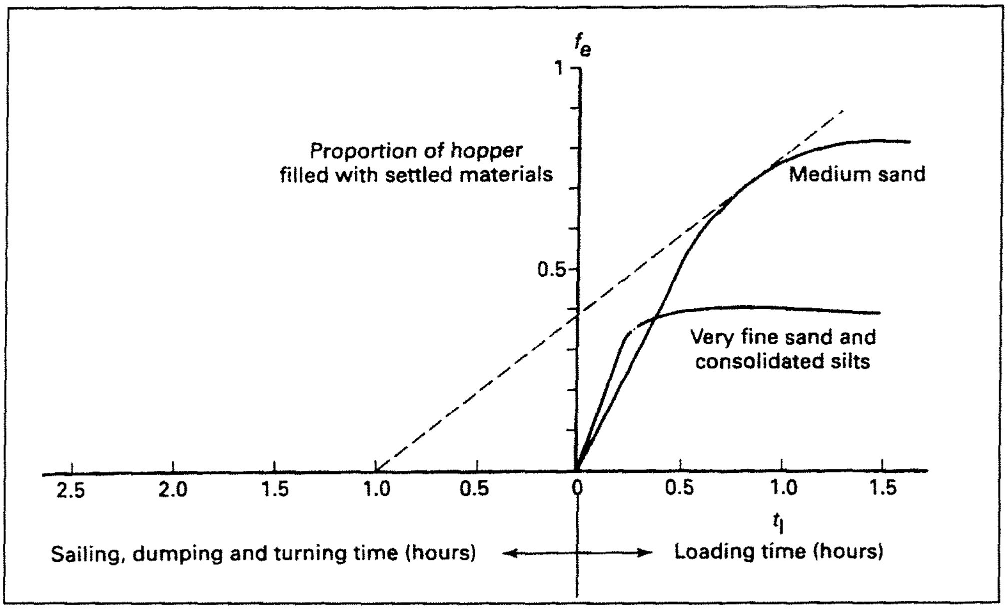

Figure 6—8 Trailing suction hopper loading graph.

[1] Since t_l is not known at the beginning of this exercise, a certain amount of iterations may be required to arrive at the final results.

The unproductive cycle time (equation (6-9)) is set off on the left hand scale. The tangent to the loading curve from this point will touch the loading curve at the time that loading should be terminated. Loading time, t_l, and hopper filling factor, f_e, can then be read off the graph. Since t_l, is not known at the beginning of this exercise, a certain amount of iterations may be required to arrive at the final result.

The total load of the hopper then becomes:

$$\text{Total load} = f_e \times H \tag{6-5}$$

Turning

The number of turns at the end of a dredging run, assuming that dredging is carried out at 2.0 knots (1 knot = 1.85 km/hr):

$$\text{Number of turns} = \frac{\text{total sailing distance while trailing}}{\text{run length}} = \frac{2 \times 1.85 \times t_l}{l} \tag{6-6}$$

The time spent turning thus becomes:

$$\text{Turning time} = \text{number of turns} \times \text{turning time} = \frac{2 \times 1.85 \times t_l \times t_t}{l} \tag{6-7}$$

Sailing

The time taken to sail to the disposal ground and return to the dredging site is given, approximately, by:

$$\text{Sailing time} = \frac{\text{distance to the disposal site}}{\text{sailing speed}} = \frac{g}{V_g} \tag{6-8}$$

Disposal

The time taken to dispose of dredged material, t_d, is known at the outset.

The output

The total period of unproductive time is derived from:

Unproductive cycle time =

$$\text{turning time} + \text{sailing time} + \text{dumping time} = \frac{3.7 \times t_l \times t_t}{l} + \frac{1.02 \times g}{V_g} + t_d \tag{6-9}$$

The loading time, t_l, and the hopper fill factor, f_e, can then be found (after 2 iterations) from the loading graph [see Figure 6—8],

	fine to medium sand	silt, very soft clay
t_l [hours]	0.75	0.25
f_e [-]	0.68	0.35

Table 6-12 Loading variables of dredging output calculation.

Maximum potential output, P_{max}, is given by $\frac{\text{total load}}{\text{total cycle time}}$

$$P_{max} = \frac{\text{total load}}{\text{total cycle time}} = \frac{H \times f_e}{B\left(t_l + \frac{3.7 t_t t_l}{l} + \frac{1.02 g}{V_g} + t_d\right)} \tag{6-10}$$

Finally, to obtain the output, P, during a period of productive and non-productive working time, reduction factors for delay, operational and mechanical breakdown have to be applied to P_{max}.

The average output, P, is obtained from:

$$P = f_d \times f_o \times f_b \times P_{max} \tag{6-11}$$

where (previous paragraph):

f_b = 0.85

f_d = 1

f_o = 0.75

In Table 6-13 the estimated output of the two dredging vessels is given. A detailed calculation is given in the Appendix D.

	Floris*	Bali*
Capital dredging	43,540	66,395
Maintenance dredging	52,107	87,066

Table 6-13 Estimated output of the two dredging vessels in m^3/week.

6.5.3 Capital dredging costs

The total capital dredging costs for three different maintenance levels are given in Table 6-14. With the capital dredging amount, given in Table 6-4, and the output of the dredging vessels, given in Table 6-13, the number of weeks needed to carry out the project can be calculated,

$$\text{weeks} = \frac{\text{total dredging amount}}{\text{output of dredging vessels}}$$

	River shoals			P.Payung / outerbar			Total
Below LWS	Weeks	Costs per week	Costs	Weeks	Costs per week	Costs	Cost
6.5	0.6	68,128	44,202	70.6	137,402	9,694,005	9,738,207
7.0	15.4		1,048,460	90.3		12,402,768	13,451,228
7.5	67.5		4,596,237	129.0		17,722,875	22,319,112

Table 6-14 Resulting capital dredging costs in Dfl.

When the project is carried out it has to be decided whether an extra dredger is needed or the duration of the project takes longer than a year, in those cases that the number of weeks, needed to execute the project ,exceeds 52 weeks.

6.5.4 Maintenance dredging costs

The total maintenance dredging costs for different maintenance levels are given in

	River shoals			P.Payung /outerbar			Total
Below LWS	Weeks	Costs per week	Costs	Weeks	Costs per week	Costs	Costs
6.5	6.1	68,128	416,449	31.9	137,402	4,389,527	4,805,976
7.0	6.9		472,098	40.6		5,584,858	6,056,956
7.5	7.5		507,942	50.7		6,961,905	7,469,847

Table 6-15 Resulting maintenance dredging costs in Dfl..

With the capital dredging amount, given in Table 6-5, and the output of the dredging vessels, given in Table 6-13, the number of weeks needed to carry out the project can be calculated,

$$\text{weeks} = \frac{\text{total dredging amount}}{\text{output of dredging vessels}}$$

	River shoals			P.Payung /outerbar			Total
Below LWS	Weeks	Costs per week	Costs	Weeks	Costs per week	Costs	Costs
6.5	6.1	68,128	416,449	31.9	137,402	4,389,527	4,805,976
7.0	6.9		472,098	40.6		5,584,858	6,056,956
7.5	7.5		507,942	50.7		6,961,905	7,469,847

Table 6-15 Resulting maintenance dredging costs in Dfl..

6.5.5 Total dredging costs

6.5.5.1 Analysis of dredging volumes and dredging costs in recent years.

Rukindo is the state dredging company that has dredged the Musi river for more than 17 years. Over the last 10 years the average dredged *in situ* volume was +/- 2.5 million $m^3_{in\ situ}$/year according to Rukindo. The dredged *in situ* volume ($m^3_{in\ situ}$) is calculated, by Rukindo, from the dredged amounts (m^3_{hopper}, which are measured by Rukindo) with the assumption that at the river shoals mainly sand is dredged and at P. Payung (river mouth) and at the outerbar mainly silt and clay are dredged.

In the following Table 6-16 the data as given by Rukindo, is analyzed.

Part of the river	in situ[1] volume ($m^3_{in\ situ}$/year)	f_e[2]	dredged volume (m^3_{hopper} /year)	Costs[3] in (Rp.x10^6)
Upstream Pulau Ayam	318,515	1 (sand)	318,515	812
River mouth	210,935	0.5 (silt)	421,870	1,075
Outerbar	1,863,724	0.5 (silt)	3,727,448	9,505
Total	2,393,173		4,467,833	11,493[4]

Table 6-16 Calculation of the amount and costs according to Rukindo.

6.5.5.2 Resulting total dredging costs

The total dredging costs per year include capital dredging and maintenance dredging for three different maintenance levels (see Table 6-17).

Total dredging costs = Maintenance dredging costs + 10 % on investment of capital dredging costs.

Below LWS	Total costs
6.5	5,974,561
7.0	7,671,103
7.5	10,148,141

Table 6-17 Resulting total dredging costs in Dfl..

6.6 Conclusions

- The actual maintenance level is LWS - 6.5 m, with the exception of the Musi river mouth and the outerbar, where the actual maintenance depth is less.

[1] The summarized dredged *in situ* volumes. A specification of the dredged *in situ* volumes, given by Rukindo, is give in the Appendix.

[2] The proportion of hopper filled according to Rukindo.

[3] Rp 2550.-/m^3 (= \$1/ m^3 US, 1996)

[4] The yearly budget of Rukindo (≈ Rp. 10 billion)

- As discussed in paragraph 6.3.2 the location of the dumping site has its influence on the yearly dredging amount. It is not known which amount of the dredged soils are dredged every year.
- Local conditions such as density current, tidal currents along the coast will have their influences and they have to be investigated to make a analysis of the siltation process at the outerbar.(6.3.2)
- Compared to the calculated proportion of hopper filled, f_e, in Table 6-12, based on Figure 6—8, the values assumed by Rukindo seem to be somewhat high. This means the dredged *in situ* volumes are probably lower.

7. Total project cost

7.1 Cost of river structures and dredging

When groynes are constructed at the river shoals the dredging costs are limited to the dredging costs at the outerbar. In Figure 7—1 the dredging costs and the dredging costs in combination with the groynes construction costs are presented.

below LWS	dredging	dredging + river works
6.5	5,974,561	6,058,612
7	7,671,103	7,699,505
7.5	10,148,141	10,140,698
8	11,737,372	11,696,420
8.5	14,228,897	14,154,435

Table 7-1 Resulting total project costs a year in Dfl.

Three different price levels for river works (see Figure 7—1) are taken into account :

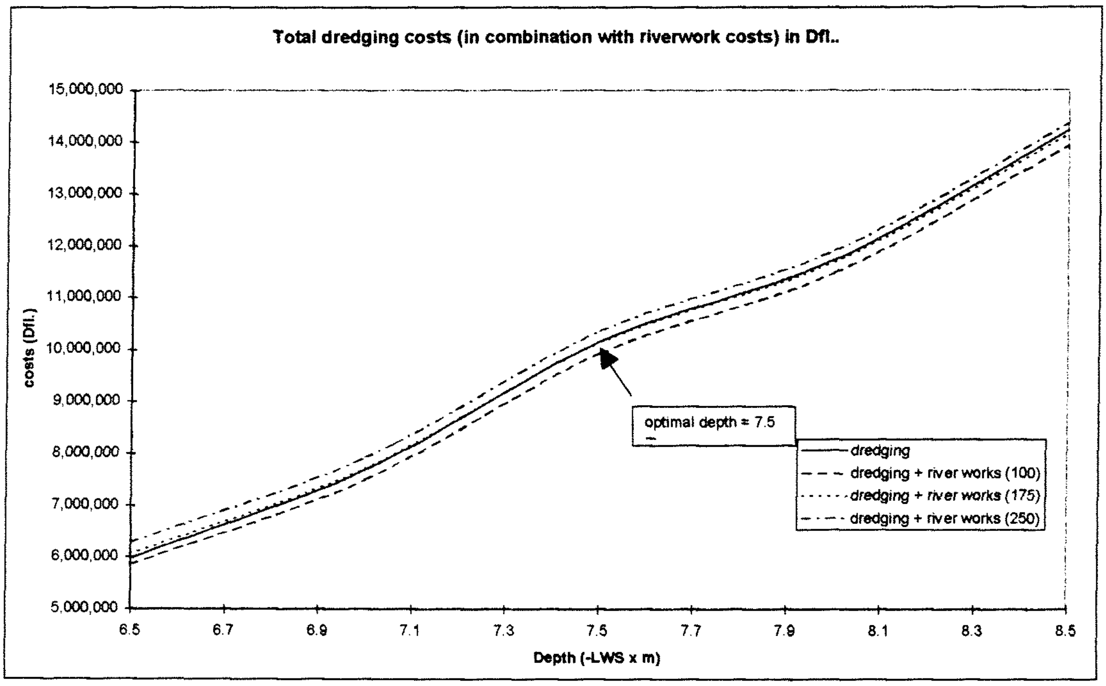

Figure 7—1 Total dredging costs (in combination with riverwork costs) in Dfl..

- Dfl. 100,- /m
- Dfl. 175,- /m

- Dfl. 250,- /m

It is assumed that due to the construction of the groynes no soils will have to be dredged on the river shoals. The advantage of applying river works is minimal because of the relatively small amount (13 % of total) of sediments dredged on the river shoals (and thus the costs) as can be seen in Figure 7—1. The application of riverworks will be profitable at a maintenance depth of the Musi river channel larger than LWS - 7.5 m when the costs of the groynes are estimated on Dfl. 175.- /m.

7.2 Summary of costs for channel optimisation

The advantage of applying river works is minimal because of the relatively small amount (see previous paragraph). Together with the involved risks of the application of river works lead to the following figure.

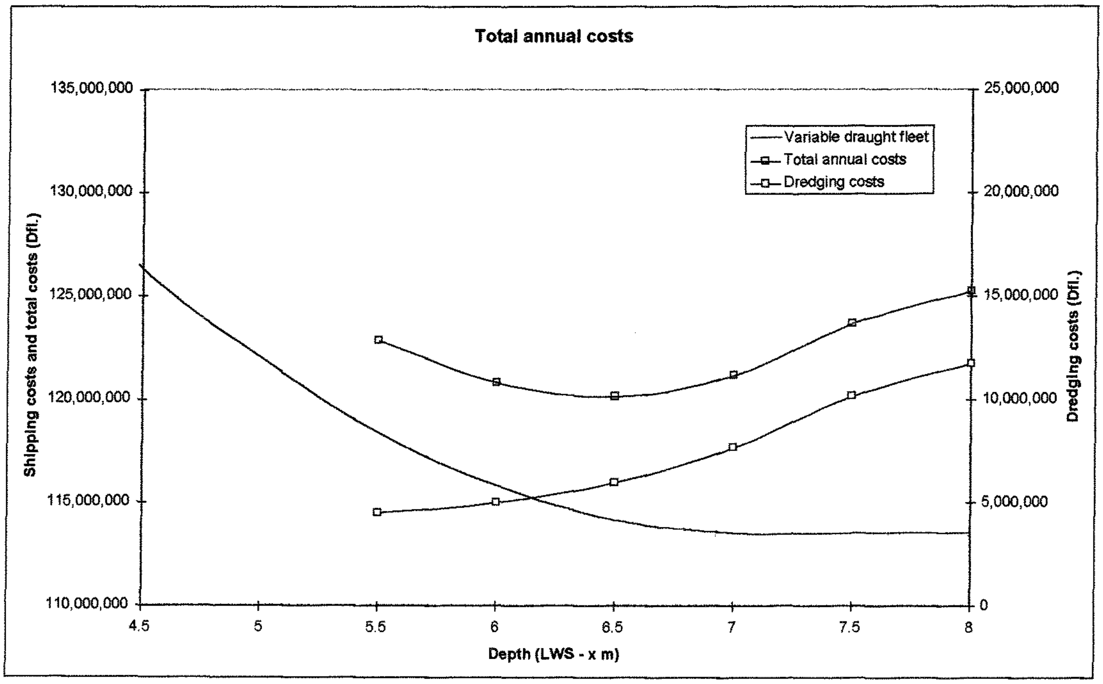

Figure 7—2 Total annual costs in Dfl.

The total costs are composed of the shipping costs calculated by R. Brans [3] and the dredging costs calculated in this study (see chapter 6). The optimal river depth of the Musi river is LWS - 6.5 m as can be seen in Figure 7—2.

The shipping costs are calculated with an exchange rate of Dfl. 1,- = Rp.1330,-

8. Final conclusions and recommendations

8.1 Conclusions

From this study the following conclusions can be drawn:

- The Musi river is strongly influenced by tidal conditions at sea. The Musi river has a significant interaction with its branches.
- The dredging process can be improved. Better dredging reports concerning dumping locations, travel distances and hopper fill properties will probably give less dredging costs.
- The actual maintenance level is LWS - 6.5 m, with the exception of the Musi river mouth and the outerbar, where the actual maintenance depth is less (LWS - 4.5 up to LWS - 6.0).
- The reduction of the dredging cost by applying riverworks is minimal, because of the relatively small amount (13 % of total) of sediments dredged on the river shoals.
- The optimum water depth in the Musi river (for a fleet with 30 % higher draughts [3]) is around LWS - 6.5 m.

8.2 Recommendations

The following recommendations are made:

- To get more insight on the siltation process on the Musi river (also resiltation of dumped soil), the dredging process more and continually performed measurements are needed.
- The DUFLOW model is a rough model and which can be improved with more and continually performed measurements.
- The location of Tg. Api Api, near the rivers mouth, is seriously considered by the large terminals from the Port of Palembang. Because of the uncertainties in a different execution of the river works a closer look at the future perspectives and developments of those large industries is needed.

8.3 Specification of required additional data

1. Sounding charts

Soundings should be made of the full river including the outer bar and not only of the dredged channel. This data can be used to optimize the Musi River Model. It shall also indicate at what places in the riverbed there is opportunity to dump dredged material.

2. Soil samples

Soil samples should be collected from the river bed and the outer bar in the areas that are dredged regularly. At least 2 samples for every km of channel are needed. These data can be used to see if material is suitable for beneficial use. It will give a more accurate indication in which locations low cost dredging methods are feasible.

3. Dredging

For one full year, all the trips made by the trailing suction hopper dredges that carry out the maintenance dredging should be listed, including the following data:

- the sector where dredging took place
- the sector where the load was dumped
- the quantity dredged each trip
- the hopper capacity of the vessel that did the dredging
- the full cycle time of each trip (time between start of dredging till start of dredging in the next trip
- the date of the trip
- the suction time
- the name of the dredge

On the basis of these results, it is possible to indicate which sectors are most expensive to dredge and where most savings can be achieved by alternative methods, as Water Injection Dredge.

9. Synthesis

In this study the possibilities of an increase of the navigational depth of the Musi river is investigated. The application of river works and dredging on shallow parts costs money. But as a result of the increase of river depth an increase of the load factor in combination with a reduction of the waiting times (R. Brans, June 1997) can be accomplished, which yields a profit.

Part of the preparation for this thesis was a two month stay at the ITS in Surabaya, Java. During that period the Musi river and the Port of Palembang were visited.

The Musi river is a tidal river in the south of Sumatra, one of the five major Indonesian islands. The Musi river is used by large ocean-going vessels to enter the Port of Palembang, an industrial city 100 km upstream. There are several bars along the river and at the outerbar. Larger vessels (normative vessel: L =160 m, D = 6.0 m) they gave to wait due to their large draught at the river mouth for high tide before they can sail the Musi river.

The following cases at different channel depths were reviewed:

- maintaining the rivers depth near the shoals through dredging only
- constricting the flow width by the application of river works to increase the equilibrium depth of the main channel which will result in a lower dredging amount

To simulate changes in the river as a result of riverworks and dredging the 'Musi River Model' has been developed with DUFLOW. The results of a simulation of a river constriction or depth increase are used to make sediment transport calculations for the different alternatives.

Until now the main channel is maintained through dredging. To estimate the amount of soil to be dredged for the different alternatives the dredging history and the sediment transport calculations are used. The riverworks play a role in this calculation as mentioned before.

The resulting costs of a depth increase are compared with the benefits due to shorter waiting times and a higher loading factor. It is find out that the present maintenance depth of LWS - 6.5 m is optimal and result in minimal total costs. How ever, the depth near the river mouth bar is less and should be deepened to match the optimum.

Steven A. Heukelom

Delft, August 29, 1997

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Appendixes

Appendix A: Bottom samples

Sediment	River : Musi				
Grain size	Date : 30 July 1996				
Location	Time	Sediment diameter (mm)			
		D90	D65	D50	Dm
Dermaga BBC	07:45	1.400	0.800	0.700	0.660
	08:05	0.940	0.750	0.650	0.600
Sungai Lais	08:10	0.950	0.690	0.570	0.450
	08:25	0.084	0.040	0.027	0.024
Sungai Borang	08:30	0.900	0.580	0.420	0.370
Muara Kumbang	09:00	0.680	0.400	0.340	0.340
	09:05	0.480	0.350	0.280	0.250
	09:15	0.425	0.230	0.200	0.240
	09:20	0.490	0.280	0.230	0.250
	09:25	0.190	0.065	0.037	0.032
Selat Jaran	09:33	0.630	0.370	0.310	0.270
	09:50	0.440	0.260	0.210	0.230
	09:55	0.370	0.220	0.190	0.210
	10:03	0.200	0.130	0.050	0.026
	10:08	0.390	0.220	0.190	0.180
	11:04	0.210	0.175	0.160	0.160
Sungai Upang	11:10	0.400	0.240	0.200	0.210
	11:25	0.450	0.290	0.230	0.250
	11:37	0.500	0.240	0.200	0.240
Pulau Ayam	11:50	0.200	0.150	0.100	0.038
	11:55	0.220	0.175	0.150	0.140
	12:00	0.240	0.190	0.170	0.140
	12:07	0.360	0.180	0.140	0.085
	12:12	0.220	0.180	0.165	0.160
	12:17	0.300	0.200	0.180	0.160
	12:25	0.125	0.050	0.034	0.034
	12:30	0.200	0.140	0.090	0.045
Parit 12	12:35	0.190	0.070	0.040	0.035
	13:08	0.125	0.042	0.020	0.019
	13:12	0.066	0.046	0.025	0.021
	13:17	0.200	0.130	0.056	0.040
	13:20	0.200	0.075	0.040	0.040
Pulau Payung	13:25	0.130	0.051	0.033	0.033
	13:35	0.500	0.300	0.220	0.240

Sediment	River : Musi				
Grain size	Date : 23 July 1996				
Location	Time	Sediment diameter (mm)			
		D90	D65	D50	Dm
Dermaga BBC	11:07	0.900	0.500	0.325	0.300
	11:45	1.000	0.725	0.600	0.550
Sungai Lais	11:53	0.900	0.660	0.600	0.500
Sungai Borang	12:10	0.180	0.046	0.025	0.020
	12:25	0.080	0.022	0.010	0.012
Muara Kumbang	12:45	0.500	0.330	0.250	0.250
	12:58	0.450	0.300	0.250	0.200
	13:07	0.400	0.225	0.200	0.180
Selat Jaran	13:18	0.450	0.290	0.225	0.240
	13:25	0.480	0.275	0.210	0.230
	13:30	0.430	0.250	0.200	0.200
	13:40	0.230	0.160	0.140	0.125
	14:35	0.460	0.360	0.320	0.290
	14:40	0.800	0.440	0.325	0.325
Sungai Upang	14:50	0.400	0.220	0.185	0.200
	15:00	0.230	0.125	0.055	0.036
Pulau Ayam	15:23	0.160	0.089	0.030	0.024
	15:30	0.240	0.180	0.160	0.150
	15:35	0.200	0.085	0.048	0.035
	15:40	0.070	0.027	0.017	0.015
	15:45	0.275	0.180	0.150	0.150
	15:55	0.240	0.180	0.160	0.150
Parit 12	17:40	0.120	0.040	0.024	0.018
	16:07	0.175	0.035	0.014	0.014
	10:15	0.160	0.035	0.017	0.015
P. Payung Kanan	16:20	0.180	0.080	0.044	0.035
P. Payung Kiri	16:40	0.050	0.010	0.006	0.008

Appendix B: History of dredging volumes

No.	Location	Year of dredging	Volume of dredging (m3)
1	Ambang Luar C1	1992/1993	842,950
		1993/1994	792,499
		1994/1995	841,547
		1995/1996	856,969
2	Ambang Luar C2	1992/1993	707,020
		1993/1994	904,695
		1994/1995	1,005,140
		1995/1996	872,872
3	Payung Utara	1992/1993	45,310
		1993/1994	273,299
		1994/1995	145,141
		1995/1996	167,452
4	Payung Barat	1992/1993	104,046
		1993/1994	53,130
		1994/1995	44,309
		1995/1996	30,912
5	Payung Selatan	1992/1993	141,335
		1993/1994	220,145
		1994/1995	82,409
		1995/1996	167,452
6	Penyebrangan Upang	1992/1993	86,451
		1993/1994	72,220
		1994/1995	55,856
		1995/1996	34,339
7	Selat Jaran	1992/1993	62,560
		1993/1994	100,688
		1994/1995	13,098
		1995/1996	40,983

No.	Location	Year of dredging	Volume of dredging (m3)
8	Muara Selat Jaran	1992/1993	168,015
9	Pulau Ayam	1992/1993	210,220
10	Ayer Kumbang	1992/1993 1993/1994 1994/1995 1995/1996	45,080 16,646 11,046 23,486
11	Sungai Lais	1992/1993 1993/1994 1994/1995 1995/1996	57,070 68,425 102,178 105,697

Appendix C: DUFLOW files

Control file

Nodes file

Network file

Initial conditions file

Boundary conditions file

*** DUFLOW data file :C:\MUSIDOCS\DUFLOW\FINAL\SIMPLE.CTR**

```
* Control data      program version: 2.02
*
TIME 960728     0 960731     0 960729     0
CONT 10.000 60.000                      1
     0.00   0.45          0    1
   1.0000     3 10.000  0.50    0     0
OUTS 23,25,1,4,7
```

```
* DUFLOW data file :C:\MUSIDOCS\DUFLOW\FINAL\SIMPLE.NOD
*  Network data     program version: 2.02
*
+FI    0.0
  1      0      0    0E+00  0.00
  2   7600   1100    0E+00  0.00
  3  12100  12150    0E+00  0.00
  4  14450  19800    0E+00  0.00
  5  18500  30000    0E+00  0.00
  6  17500  41500    0E+00  0.00
  7  16250  52000    0E+00  0.00
  8  15250  64250    0E+00  0.00
  9  13250  68500    0E+00  0.00
 10  14250  70750    0E+00  0.00
 11  16050  91250    0E+00  0.00
 12   7500  25000    0E+00  0.00
 13   7500  60000    0E+00  0.00
 14  25000  75000    0E+00  0.00
 15  16000  46500    0E+00  0.00
 16  18000  46500    0E+00  0.00
 17   7600   6000    0E+00  0.00
 18  12150   4000    0E+00  0.00
 19  18000  35000    0E+00  0.00
 20 -200000 -200000  0E+00  0.00
 21 -195000 -195000  0E+00  0.00
 22 -50000  -50000   0E+00  0.00
```

23	24000	68846	0E+00	0.00
24	14750	67500	0E+00	0.00
25	19208	35000	0E+00	0.00
30	-10000	-10000	0E+00	0.00
31	-20000	-20000	0E+00	0.00
32	-30000	-30000	0E+00	0.00
33	-40000	-40000	0E+00	0.00
40	-180000	-180000	0E+00	0.00
41	-165000	-165000	0E+00	0.00
42	-150000	-150000	0E+00	0.00
43	-135000	-135000	0E+00	0.00
44	-120000	-120000	0E+00	0.00
45	-105000	-105000	0E+00	0.00
46	-90000	-90000	0E+00	0.00
47	-75000	-75000	0E+00	0.00
48	-60000	-60000	0E+00	0.00

*** DUFLOW data file :C:\MUSIDOCS\DUFLOW\FINAL\SIMPLE.NET**

```
*   Network data      program version: 2.02
*

* DUFLOW data file :C:\MUSIDOCS\DUFLOW\FINAL\SIMPLE1.NET
*   Network data      program version: 2.02
*

SECT  1     1    1    2  7679  -4.70  -5.41  50.00  50.00
W        262.0   3.6
H   0.0000 4.0000 15.000
BS  250.00 500.00 500.00
BB  252.00 505.00 505.00

SECT  2     2    2   17  11931  -3.68  -3.76  50.00  50.00
W        202.0   3.6
H   0.0000 4.0000 15.000
BS  300.00 300.00 300.00
BB  302.00 302.00 302.00

SECT  3     3    3    4  8003 -12.93  -5.25  50.00  50.00
W        197.0   3.6
H   0.0000 4.0000 15.000
BS1 400.00 400.00 400.00
BS2 450.00 450.00 450.00
BB1 404.00 404.00 404.00
BB2 452.00 452.00 452.00

SECT  4     4    4    5  10975  -5.25  -7.46  50.00  50.00
W        202.0   3.6
H   0.0000 4.0000 15.000
BS  800.00 800.00 800.00
BB  802.00 802.00 802.00

SECT  5     5    5   19  5025  -7.46  -5.48  50.00  50.00
W        174.0   3.6
H   0.0000 4.0000 15.000
BS  500.00 500.00 500.00
BB  502.00 502.00 502.00

SECT  6     6    6   15  10574  -9.73  -2.31  50.00  50.00
W        173.0   3.6
```

```
H   0.0000 4.0000 15.000
BS  150.00 150.00 150.00
BB  152.00 152.00 152.00

SECT  7      7    7    8  12291 -14.98  -3.43  50.00  50.00
W        175.0   3.6
H   0.0000 4.0000 15.000
BS1  150.00 150.00 150.00
BS2  600.00 600.00 600.00
BB1  152.00 152.00 152.00
BB2  602.00 602.00 602.00

SECT  8      8    8    9  4697  -3.43 -10.62  50.00  50.00
W        155.0   3.6
H   0.0000 4.0000 15.000
BS1  400.00 400.00 400.00
BS2  600.00 600.00 600.00
BB1  402.00 402.00 402.00
BB2  602.00 602.00 602.00

SECT  9      9    9   10  2462 -10.62  -4.67  50.00  50.00
W        204.0   3.6
H   0.0000 4.0000 15.000
BS1  550.00 550.00 550.00
BS2  300.00 300.00 300.00
BB1  552.00 552.00 552.00
BB2  302.00 302.00 302.00

SECT  10     10   10   11  20579  -4.67 -15.00  50.00  50.00
W        185.0   3.6
H   0.0000 4.0000 25.000
BS1  900.00 900.00 900.00
BS2  5000.0 5000.0 5000.0
BB1  902.00 902.00 902.00
BB2  5002.0 5002.0 5002.0

SECT  11     11    8   24  3288  -2.77  -4.50  50.00  50.00
W        171.0   3.6
H   0.0000 4.0000 15.000
BS  200.00 200.00 200.00
BB  202.00 202.00 202.00

SECT  12     12    4   12  5411  -6.00  -6.00  50.00  50.00
```

```
W          122.0   3.6
H    0.0000 4.0000 15.000
BS   100.00 100.00 100.00
BB   102.00 102.00 102.00

SECT  13      13    12    13  45000  -4.50  -4.00  50.00  50.00
W          180.0   3.6
H    0.0000 4.0000 15.000
BS   250.00 250.00 250.00
BB   252.00 252.00 252.00

SECT  14      14    13     9  10262  -4.00  -4.00  50.00  50.00
W          214.0   3.6
H    0.0000 4.0000 15.000
BS1  300.00 300.00 300.00
BS2  700.00 700.00 700.00
BB1  302.00 302.00 302.00
BB2  702.00 702.00 702.00

SECT  15      15    15     7   5506  -2.31  -9.69  50.00  50.00
W          183.0   3.6
H    0.0000 4.0000 15.000
BS   150.00 150.00 150.00
BB   152.00 152.00 152.00

SECT  16      16     6    16   5025  -9.79  -9.89  50.00  50.00
W          186.0   3.6
H    0.0000 4.0000 15.000
BS   200.00 200.00 200.00
BB   202.00 202.00 202.00

SECT  17      17    16     7   5772  -9.89 -14.98  50.00  50.00
W          162.0   3.6
H    0.0000 4.0000 15.000
BS   200.00 200.00 200.00
BB   202.00 202.00 202.00

SECT  18      18    17     3   7621  -3.76  -6.26  50.00  50.00
W          216.0   3.6
H    0.0000 4.0000 15.000
BS   300.00 300.00 300.00
BB   302.00 302.00 302.00
```

```
SECT 19     19    2   18  5396  -5.41  -5.66  50.00  50.00
W         237.0   3.6
H   0.0000 4.0000 15.000
BS   450.00 450.00 450.00
BB   452.00 452.00 452.00

SECT 20     20   18    3  8150  -5.66 -12.93  50.00  50.00
W         180.0   3.6
H   0.0000 4.0000 15.000
BS   550.00 550.00 550.00
BB   552.00 552.00 552.00

SECT 21     21   19    6  6519  -5.48  -9.79  50.00  50.00
W         176.0   3.6
H   0.0000 4.0000 15.000
BS   500.00 500.00 500.00
BB   502.00 502.00 502.00

SECT 22     22   25   23 34184  -5.50  -6.00  50.00  50.00
W         188.0   3.6
H   0.0000 4.0000 15.000
BS1  450.00 450.00 450.00
BS2  900.00 900.00 900.00
BB1  452.00 452.00 452.00
BB2  902.00 902.00 902.00

SECT 23     23   20   21  7071   7.00   6.50  50.00  50.00
W         225.0   3.6
H   0.0000 4.0000 15.000
BS   100.00 100.00 100.00
BB   102.00 102.00 102.00

SECT 24     24   48   22 14142  -2.50  -3.50  50.00  50.00
W         225.0   3.6
H   0.0000 4.0000 15.000
BS   150.00 150.00 150.00
BB   152.00 152.00 152.00

SECT 25     25   22   33 14142  -2.50  -2.94  50.00  50.00
W         225.0   3.6
H   0.0000 4.0000 15.000
BS   150.00 150.00 150.00
BB   152.00 220.00 220.00
```

```
SECT 26    26   23   14  6235 -6.00 -6.50 50.00 50.00
W       189.0   3.6
H   0.0000 4.0000 15.000
BS1  900.00 900.00 900.00
BS2  1000.0 1000.0 1000.0
BB1  902.00 902.00 902.00
BB2  1002.0 1002.0 1002.0

SECT 27    27   24   10  3288 -4.50 -5.86 50.00 50.00
W       171.0   3.6
H   0.0000 4.0000 15.000
BS   200.00 200.00 200.00
BB   202.00 202.00 202.00

SECT 28    28    5   25  5050 -5.00 -5.50 50.00 50.00
W        32.0   3.6
H   0.0000 4.0000 15.000
BS1  400.00 400.00 400.00
BS2  450.00 450.00 450.00
BB1  402.00 402.00 402.00
BB2  452.00 452.00 452.00

SECT 30    30   30    1 14142 -4.26 -4.70 50.00 50.00
W       225.0   3.6
H   0.0000 4.0000 15.000
BS   150.00 430.00 430.00
BB   152.00 500.00 500.00

SECT 31    31   31   30 14142 -3.82 -4.26 50.00 50.00
W       225.0   3.6
H   0.0000 4.0000 15.000
BS   150.00 360.00 360.00
BB   152.00 430.00 430.00

SECT 32    32   32   31 14142 -3.38 -3.82 50.00 50.00
W       225.0   3.6
H   0.0000 4.0000 15.000
BS   150.00 290.00 290.00
BB   152.00 360.00 360.00

SECT 33    33   33   32 14142 -2.94 -3.38 50.00 50.00
W       225.0   3.6
```

```
H   0.0000 4.0000 15.000
BS  150.00 220.00 220.00
BB  152.00 290.00 290.00

SECT 40     40   21   40 21213  5.50  5.50 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
BS  150.00 150.00
BB  152.00 152.00

SECT 41     41   40   41 21213  4.25  4.25 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
BS  150.00 150.00
BB  152.00 152.00

SECT 42     42   41   42 21213  3.00  3.00 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
BS  150.00 150.00
BB  152.00 152.00

SECT 43     43   42   43 21213  1.75  1.75 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
BS  150.00 150.00
BB  152.00 152.00

SECT 44     44   43   44 21213  0.50  0.50 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
BS  150.00 150.00
BB  152.00 152.00

SECT 45     45   44   45 21213 -0.75 -0.75 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
BS  150.00 150.00
BB  152.00 152.00

SECT 46     46   45   46 21213 -1.50 -1.50 50.00 50.00
W        225.0  3.6
H   0.0000 10.000
```

```
BS   150.00 150.00
BB   152.00 152.00

SECT  47      47    46    47  21213  -2.00  -2.00  50.00  50.00
W          225.0    3.6
H    0.0000 10.000
BS   150.00 150.00
BB   152.00 152.00

SECT  48      48    47    48  21213  -2.50  -2.50  50.00  50.00
W          225.0    3.6
H    0.0000 10.000
BS   150.00 150.00
BB   152.00 152.00
```

```
* DUFLOW data file :C:\MUSIDOCS\DUFLOW\FINAL\SIMPLE.BEG
* Flow  Initial conditions      program version: 2.02
*
  1 1.9000 1.9000 0.0000 0.0000
  2 1.9000 1.9000 0.0000 0.0000
  3 1.9000 1.9000 0.0000 0.0000
  4 1.9000 1.9000 0.0000 0.0000
  5 1.9000 1.9000 0.0000 0.0000
  6 1.9000 1.9000 0.0000 0.0000
  7 1.9000 1.9000 0.0000 0.0000
  8 1.9000 1.9000 0.0000 0.0000
  9 1.9000 1.9000 0.0000 0.0000
 10 1.9000 1.9000 0.0000 0.0000
 12 1.9000 1.9000 0.0000 0.0000
 13 1.9000 1.9000 0.0000 0.0000
 14 1.9000 1.9000 0.0000 0.0000
 15 1.9000 1.9000 0.0000 0.0000
 16 1.9000 1.9000 0.0000 0.0000
 17 1.9000 1.9000 0.0000 0.0000
 18 1.9000 1.9000 0.0000 0.0000
 19 1.9000 1.9000 0.0000 0.0000
 20 1.9000 1.9000 0.0000 0.0000
 21 1.9000 1.9000 0.0000 0.0000
 22 1.9000 1.9000 0.0000 0.0000
 23 9.5000 9.0000 0.0000 0.0000
 24 9.0000 1.9000 0.0000 0.0000
```

25 1.9000 1.9000 0.0000 0.0000

11 1.9000 1.9000 0.0000 0.0000

26 1.9000 1.9000 0.0000 0.0000

27 1.9000 1.9000 0.0000 0.0000

28 1.9000 1.9000 0.0000 0.0000

30 1.9000 1.9000 0.0000 0.0000

31 1.9000 1.9000 0.0000 0.0000

32 1.9000 1.9000 0.0000 0.0000

33 1.9000 1.9000 0.0000 0.0000

40 9.0000 8.0000 0.0000 0.0000

41 8.0000 7.0000 0.0000 0.0000

42 7.0000 6.0000 0.0000 0.0000

43 6.0000 5.0000 0.0000 0.0000

44 5.0000 4.0000 0.0000 0.0000

45 4.0000 3.0000 0.0000 0.0000

46 3.0000 2.0000 0.0000 0.0000

47 2.0000 2.0000 0.0000 0.0000

48 2.0000 2.0000 0.0000 0.0000

```
* DUFLOW data file :C:\MUSIDOCS\DUFLOW\FINAL\SIMPLE.BND
* Flow  Bound. cond./struct ctrl.     program version: 2.02
*
Q                                        20
P   300.00
H           60 960728    0                     11
   2.0000 1.9000 1.8000 1.7000 1.6000 1.4000
   1.2000 1.1000 .90000 .80000 .90000 1.0000
   1.2000 1.6000 2.0000 2.4000 2.7000 3.0000
   3.1000 3.1000 3.0000 2.8000 2.6000 2.3000
   2.1000 1.9000 1.8000 1.7000 1.6000 1.5000
   1.3000 1.1000 1.0000 .80000 .80000 .80000
   1.0000 1.3000 1.7000 2.1000 2.6000 2.9000
   3.2000 3.3000 3.2000 3.0000 2.8000 2.5000
   2.2000 2.0000 1.8000 1.7000 1.6000 1.5000
   1.4000 1.3000 1.1000 .90000 .80000 .70000
   .80000 1.0000 1.4000 1.8000 2.3000 2.7000
   3.1000 3.3000 3.3000 3.2000 2.9000 2.6000
   2.3000
H           60 960728    0                     14
   2.0000 1.9000 1.8000 1.7000 1.6000 1.4000
   1.2000 1.1000 .90000 .80000 .90000 1.0000
   1.2000 1.6000 2.0000 2.4000 2.7000 3.0000
   3.1000 3.1000 3.0000 2.8000 2.6000 2.3000
   2.1000 1.9000 1.8000 1.7000 1.6000 1.5000
   1.3000 1.1000 1.0000 .80000 .80000 .80000
```

1.0000 1.3000 1.7000 2.1000 2.6000 2.9000

3.2000 3.3000 3.2000 3.0000 2.8000 2.5000

2.2000 2.0000 1.8000 1.7000 1.6000 1.5000

1.4000 1.3000 1.1000 .90000 .80000 .70000

.80000 1.0000 1.4000 1.8000 2.3000 2.7000

3.1000 3.3000 3.3000 3.2000 2.9000 2.6000

2.3000

Appendix D: Dredging cost calculation

The method described is to calculate the operational costs and the production costs. Calculations are made for the two hopper dredgers (Floris* and Bali*). For each dredger the costs are calculated for capital dredging and maintenance dredging.

References to formulas in Chapter 6 are made in the column *formula*.

The four different situations which have been calculated make use of a different bulking factor, B, based on the measurements, i.e.

$$B = \frac{\text{dredged volume}}{\text{in situ volume}}$$

location	maintenance	capital	B	soil type
river shoals	x		1.1	sand, soft
		x	1.2	sand, soft to hard
P. Payung, outerbar	x		1.25	silts and clay soft to very soft
		x	1.05	clay soft to firm

Maintenance dredging, Floris* (LWS -7.5)

Calculation of average output		**formula**			
number of turns=		(6-6)	1.39		
loading time =	tl		0.75	hr	
length of dredging area=	l		2	km	
dredging speed =			2.0	knots	
1 knot =			1.853	km/hr	
turning time, one turn	tt		0.066	hr	
turning time=	A	(6-7)	**0.09**	hr	
sailing speed =	Vg		17	km/hour	
distance to disposal site =	g		10	km	
sailing time=	B	(6-8)	**0.60**	hr	
dumping time =	td		**0.083**	hr	
dumping time=	C				
For the determination of fe and tl two iterations have been made in figure .. (medium sand)					
loading time =	tl		0.75	hr	
proportion of					
hopper filled =	fe		0.68		
2 iteration					
unproductive cycle time=	A+B+C	(6-9)	**0.77**	hr	
hopper capacity =	H		2400	m^3	
hopper filled =	fe		0.68		
total load =	D	(6-5)	**1632**	m^3	
bulking factor =	B		1.1		
unproductive cycle time=	A+B+C	(6-9)	0.77	hr	
loading time =	tl		0.75	hr	
total cycle time =	E		**1.68**	hr	
maxinal potential output =	Pmax = D+E	(6-10)	**973**	m^3/hr	
traffic factor	ft	(6-3)	1	there are no losses due to traffic	
total working time available =			12		
time lost due to traffic during working =			0		
working factor	fw	(6-4)	1	there are no losses due to bad weather	
total of days when weather					
is suitable for working =			1		
total number of days			1		
delay factor =	fd =ft * fw	(6-2)	**1**		
operational factor =	fo		**0.75**	average managenent/crew	
mechanical breakdown factor =	fb		**0.85**		
average output =	P	(6-11)	**620.3**	m^3/hr	
			7,444	m^3/day	12 hour
			52,107	m^3/week	84 hour

Total cost calculation					
total amount to be dredged a year =	V		388,492	m^3	river shoal
soil = medium sand					
1 week			84	hour	
total dredging time =	T		626.3	hr	
	=		7.5	week	
number expatriate empl.			7		
			4,500	Dfl./pers/week	
number local empl.			7		
			1,125	Dfl./pers/week	
crew costs/week =			39,375	Dfl./week	
total crew costs =	Nc =		**293,567**	Dfl.	
Dredging time =	T		626.3	hours	
power dregding pumps =	Pdp		1,150	horse power	
power propulsion =	Pp		2,200	horse power	
1 HP			0.7355	kW	
consumption =			0.20	liter / kWhour	
cost =			0.50	Dfl. / litre	
dredging as part of total cycle time			0.45		
fuel costs a week			16,769	Dfl. /week	
fuel costs=	Fc		**125,025**	Dfl.	
depreciation / interest	D + I		104,613	Dfl. / year	based on 30 weeks a year, 84 hour/week
maintenance + repaire	M + R		43,642	Dfl. / year	
depreciation =			2,851	Dfl. / week	
depreciation =	D		**148,255**	Dfl.	
standard value	N		32,090,000	Dfl.	
			0.04	%	
insurance			246.85	Dfl. / week	
insurance	I		**12,836**	Dfl.	
overhead\profit\risk			8,886	Dfl./week	
overhead\profit\risk	O		**86,952**	Dfl.	15 % of costs
total costs			68,128	Dfl./week	
total costs	TC = Nc+Fc+D+I+O		**507,942**	Dfl.	

Maintenance dredging, Bali* (LWS -7.5)

Calculation of average output		formula			
number of turns/hour =		(6-6)	0.46		
loading time =	tl		0.25	hr	
length of dredging area=	l		2	km	
dredging speed =			2.0	knots	
1 knot =			1.853	km/hr	
turning time, one turn	tt		0.066	hr	
turning time=	A	(6-7)	**0.03**	hr	
sailing speed =	Vg		17	km/hour	
distance to disposal site =	g		10	km	
sailing time=	B	(6-8)	**0.60**	hr	
dumping time =	td		**0.083**	hr	
dumping time=	C				
For the determination of fe and tl two iterations have been made in figure .. (medium sand)					
loading time =	tl		0.25	hr	
proportion of					
hopper filled =	fe		0.35		
2 iteration					
unproductive cycle time=	A+B+C	(6-9)	**0.71**	hr	
hopper capacity =	H		4700	m^3	
hopper filled =	fe		0.35		
total load =	D	(6-5)	**1645**	m^3	
bulking factor =	B		1.05		
unproductive cycle time=	A+B+C	(6-9)	0.71	hr	
loading time =	tl		0.25	hr	
total cycle time =	E		**1.01**	hr	
maxinal potential output =	Pmax = D+E	(6-10)	**1626**	m^3/hr	
traffic factor	ft	(6-3)	1	there are no losses due to traffic	
total working time available =			12		
time lost due to traffic during working =			0		
working factor	fw	(6-4)	1	there are no losses due to bad weather	
total of days when weather					
is suitable for working =			1		
total number of days			1		
delay factor =	fd =ft * fw	(6-2)	**1**		
operational factor =	fo		**0.75**	average managenent/crew	
mechanical breakdown factor =	fb		**0.85**		
average output =	P	(6-11)	**1036.5**	m^3/hr	
			12,438	m^3/day	
			87,066	m^3/week	

Total cost calculation					
total amount to be dredged a year =	V		4,411,508	m^3	
soil = medium sand					
1 week			84	hour	
total dredging time =	T		4,256.1	hr	
	=		50.7	week	
number expatriate empl.			14		
			4,500	Dfl./pers/week	
number local empl.			14		
			1,125	Dfl./pers/week	
crew costs/week =			78,750	Dfl./week	
total crew costs =	Nc =		**3,990,128**		
dreding time =	T		4,256.1	hours	
power dreding pumps =	Pp		1,950	horse power	
power propulsion =	Pp		5,300	horse power	
1 HP			0.7355	kW	
consumption =			0.20	liter / kWhour	
cost =			0.50	Dfl. / litre	
dredging as part of total cycle time			0.25		
fuel costs a week			**35,721**	Dfl. /week	
fuel costs=	Fc		**610,426**	Dfl.	
depreciation / interest	D + I		177,246	Dfl. / year	
maintenance + repaire	M + R		61,438	Dfl. / year	
depreciation =			4,590	Dfl. / week	
depreciation =	D		**238,684**	Dfl.	
standard value	N		54,370,000	Dfl.	
			0.04	%	
insurance			418.23	Dfl. / week	
insurance	I		**21,748**	Dfl.	
overhead\profit\risk			17,922	Dfl./week	
overhead\profit\risk	O		**729,148**	Dfl.	
total costs			137,402	Dfl./week	
total costs	TC = Nc+Fc+D+I+O		**6,961,905**	Dfl.	

Nederlandse samenvatting

In deze studie is gekeken naar een aantal mogelijkheden om de vaargeul van de rivier de Musi te verdiepen. Het aanleggen van rivierwerken en het baggeren van ondiepten kost geld, echter een hogere beladingsgraad in combinatie met een kortere wachttijd (R. Brans, juni 1997) levert geld op.

Ter voorbereiding werd tijdens het verblijf van twee maanden aan het ITS, i.e. universiteit van Surabaya, Java, een bezoek gebracht aan de Musi en de haven van Palembang.

De Musi is een getijderivier in het zuiden van Sumatra, een van de vijf grote eilanden van Indonesië. De rivier vormt de toegang tot de 100 km stroomopwaarts gelegen haven van de industriestad Palembang. In de monding en in de rivier de Musi bevinden zich een aantal ondiepten. Vanwege de te grote diepgang moeten de grotere schepen (maatgevende schip: L = 160 m, D = 6.0 m) voor de monding wachten tot zij met het opkomende tij de rivier op kunnen varen.

Er is uitgegaan van de volgende alternatieven:

- verdiepen van de ondiepten door middel van baggeren
- versmallen van de Musi door middel van rivier werken, zodat er door een dieper evenwichtsprofiel minder onderhoudsbaggerwerk nodig is.

Om de gevolgen van toekomstige ingrepen in de rivier te kunnen analyseren is met behulp van DUFLOW het 'Musi River Model' gemaakt. De resultaten van de simulaties van het verdiepen en/of versmallen zijn gebruikt om berekeningen van het sedimententransport te kunnen maken voor de verschillende alternatieven.

Tot nu wordt door jaarlijks baggeren de vaargeul van de Musi op een constante diepte gehouden. Om het bagger bezwaar te bepalen voor de verschillende alternatieven zijn deze baggergegevens en de sedimententransport berekeningen gebruikt. Hiervoor is ook de aanleg van eventuele rivierwerken van belang.

De resulterende kosten van een verdieping zijn vergeleken met de opbrengsten ten gevolgen van de kortere wachttijden en een hogere beladingsgraad. Gebleken is dat de huidige onderhoudsdiepte van LWS -6,5 m een optimale diepte is waarbij de totale kosten minimaal zijn. Deze diepte wordt nu echter in de monding van de Musi niet gehaald en deze drempel zou dus op deze diepte moeten worden gebracht.

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Delft, 29 augustus 1997