A study on uncertainty in the design of river training works

and the development of a method and instruments to improve a full and consistent use of probabilistic design methods

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Royal Dutch Consulting
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PREFACE

This MSc-thesis is the result of a study performed during a six months period from April 1994 to October 1994 in Bangkok, Thailand. During this period I was given the opportunity to participate in a project of HASKONING, Royal Dutch Engineers and Architects. This project, the Mekong River Bank Erosion Study[1], was being done under the 'umbrella name' of NEDECO (Netherlands Engineering Consultants) in co-operation with two local engineering consultants, Span and WDC (Water Development Consultants).

I would especially like to thank Ir. H.J. Opdam, head of division Transport, Ports, Coastal and River Development of HASKONING and Mr. F. Carvajal Monar, M.Sc., project manager of the Mekong River Bank Erosion Study, for creating the possibility to participate in this project and providing financial support.

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Chris Verstegen
Delft, January 1995.

[1] Sometimes referred to as the Mekong River Bank *Protection* Study

ABSTRACT

The objective of this study is to make a study on dealing with uncertainty in the design of river training works. Based on this study, a method and several instruments have been developed that aim to improve a full and consistent use of a probabilistic design approach.

This study is divided into three parts. The outline is rather unconventional for a civil engineering thesis and some of the subjects treated are more related to Information Logistics and Data Management than to typical constructional aspects of civil engineering structures. But since information has become a vital asset for almost every company, in particular for an engineering consultant, it seems a very relevant aspect to study.

The first section is a general introduction to the types of uncertainty that have to be dealt with in the design of engineering structures and to the method that is used to incorporate these uncertainties: the probabilistic design approach. In a brief discussion of this approach, two of the main constraints for this approach are introduced:

1. One of the main difficulties is the statistical description of parameters in case of limited available data. These descriptions are used in the probabilistic calculations. If hardly any information is available or only a 'practical' design model is available, does it make sense to apply a probabilistic design approach?

2. Another important aspect is that probabilistic calculations and sensitivity analyses can be very time-consuming. Because many design studies have a limited time-schedule, this can prevent a full use of these methods. Is this inevitable or could certain methods and design instruments be developed that improve this applicability?

In the second section part of the first question is analysed by comparing the design results of three similar projects, in which river training works were designed. Two of these projects used a probabilistic design approach, the third one a deterministic approach. In a deterministic approach the safety is not quantified, in a probabilistic design approach, the probabilities of failure of all components of the structure are explicitly determined. The design dimensions of the structures designed in the three projects were likely to be different because of differences in natural conditions, required safety and the design models used. The objectives of this section are to study how the probabilistic design method was applied in the two projects, to study how the uncertainty of the parameters were determined and to quantify the influence of the differences in natural conditions, required safety and used models on the differences in the final designs.

In the third section, the design process of an ongoing project is analysed in order to study the second question posed in section I. The ongoing project is the Mekong River Bank Erosion Study which will be used as a reference and case study. The objective of this section is to develop a method and instruments to improve the incorporation of uncertainty in the design. First the methods of determining the uncertainties of observed and derived parameters are studied. Next, the way in which the information about these parameters is managed, communicated and used is discussed. Finally, a design model is proposed in which probabilistic calculations can be made on every required level of detail, using one single software program.

ABBREVIATIONS AND NOTATIONS USED

α	angle of bank to horizontal
A	area of flow
B	width of channel at water surface
C	roughness coefficient in Chézy formula
D	characteristic particle size
ϕ	angle of internal friction of soil
F	fetch length
g	acceleration due to gravity
h	average depth of the flow
H_s	significant wave height
i	hydraulic gradient
kg	kilogram
km	kilometres
km^2	square kilometres
kN	kilonewton
K_s	slope correction factor
m	meter
MCA	Multi Criteria Analysis
m/s	meters per second
m^2	square meters
m^3	cubic meters
mm	millimetres
ν	kinematic viscosity
N	Newton
n	roughness coefficient in Manning formula
NEDECO	Netherlands Engineering Consultants
PWD	Public Works Department of Thailand
Q	discharge
θ or Ψ	shields parameter
R	hydraulic radius
ρ_s	specific weight of stone
ρ_w	specific weight of water
s	second
τ	shear stress
u	water velocity
u*	shear velocity
WDC	Water Development Consultants
yr.	year
PDF	probability density function
CDF	cumulative distribution function

GLOSSARY

Bathymetry	Topography of sea/estuary/lake bed.
Correlation	Linear dependence between variables
Deterministic	The term deterministic indicates that the uncertainty associated with a given value or variable is not quantified
Falling apron	Layer of stone, concrete or other material to protect the toe of a structure against scour. Also referred to as launching apron.
Fetch	Direct horizontal distance (in direction of the wind) over which wind generates waves.
Gabions	Rectangular or tubular baskets made from steel wire or polymer mesh and subsequently filled with stones.
Geotextile	Permeable synthetic fabric used in conjunction with soil for the function of filtration, separation, drainage, soil reinforcement or erosion protection.
Hydraulic loads	Forces due to action of water; may be hydrodynamic or hydrostatic.
Morphology	Science of form and structure of, for example, a river channel
Open stone asphalt	Under-filled mix of mastic and stone in which the mastic connects and coats the stone. Because of its open structure, it should not be permanently placed under water, but it is a suitable material in the wave-attack zone.
Return period	In statistical analysis an event with a return period of N years is likely, on average, to be exceeded only once every N years
Revetment	A cladding of stone concrete or other material used to protect the sloping surface of an embankment, natural coast or shoreline against erosion.
Rip-rap	Randomly placed, loose rock armour, (sharp edged)
Scour	Removal of soil particles by current, propeller or wave-induced shear forces. Scour commonly refers to localised erosion of bed material.
Significant wave	Statistical term relating to the average of the highest one third of the waves of a given wave record.
Stationary process	A process in which the mean statistical properties do not vary with time.
Stochastic	Having random variation in statistics.
Suspended load	The material moving in suspension in a fluid, kept up by the upward components of the turbulent currents or by the colloidal suspension.
Turbulence	Random, very short-term fluctuations in fluid velocity. Degree of turbulence is measured by the root mean square of the fluctuations from the mean.

SECTION I

General Introduction

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1. Introduction

One of the main problems in the design of engineering structures is to deal with all the uncertainties surrounding the strength-parameters of the structure's components, the load-parameters imposed on the structure and the models that provide the relation between the strength and the loads. Although a method is available which aims to incorporate these uncertainties, the probabilistic design approach, the experience of some engineers have raised questions about the applicability of this methodology in some practical situations. These questions mainly focus on the problem of finding a reliable basis to quantify uncertainty of all the parameters if no observations are available and the time-consuming aspect of taking every possible kind of uncertainty into account in the analysis and design process.

2. Objectives of the study

The objective of this study is to make an analysis of the way in which uncertainty is incorporated in the design of several selected projects. This will be done in section II, in which the design results of three finalised projects are analysed. Based on the results of this analysis, some design methods and instruments will be developed that aim to alleviate encountered problems and improve the applicability of the probabilistic design approach. This will be done in section III, where an ongoing project, the Mekong River Bank Erosion Study, will be used as a case study for these design instruments. Although the methodology and the essential elements of the instruments could be used in the design of various engineering structures, this study focuses on the design of river training works.

3. Uncertainty

Since it is one of the main subjects of this study, the notion 'uncertainty' should be defined. There are many different types, or categories, of uncertainty and different definitions can be given of these categories. One of the most classic definitions is provided by Benjamin and Cornell (1970). They claim there are three types of uncertainty:

I. The first type is the **natural uncertainty** also indicated as the intrinsic or inherent uncertainty. It is associated with the stochastic nature in time or in space of the considered phenomenon. Some parameters used in calculations, however, do not behave stochastically in time or space but their actual state of nature is simply not (yet) known to the engineer. Although theoretically incorrect, it proves to be useful to represent this uncertainty as well with a probability function. This natural uncertainty is the uncertainty we want to model by representing it with a probability function.

II. The second type is the **statistical uncertainty**. The essential notion of this type of uncertainty is that it is impossible to determine the parameters of a distribution model with high precision, based on a finite set of observations. If the sample size becomes larger, the sample moments will approximate the distribution parameters more accurately, providing the model is correct (see next type of uncertainty).

III. The third type is **model uncertainty**. Probability distribution models are used to represent the occurrence of natural phenomena. In these models, assumptions are made about the underlying physical mechanisms, which are not necessarily correct or fully complete. The occurrence of so-called 'outliers' can be the result of relatively rare and unidentified mechanisms.
Models that represent a functional relationship between variables are also subject to model uncertainty. This is illustrated by the scatter around the model's graph and is expressed in the model as a multiplicative or additional error.

Additional to these three types we could argue there is a fourth type:

IV. Even if the sample size is large and the model is correct, discrepancies can occur because of the accuracy of the observations. This **observation uncertainty** is mainly caused by the quality of the method and the equipment used for the observations, and the condition and skill of the observer. These are factors that are hardly ever easily quantifiable but which should not be forgotten.

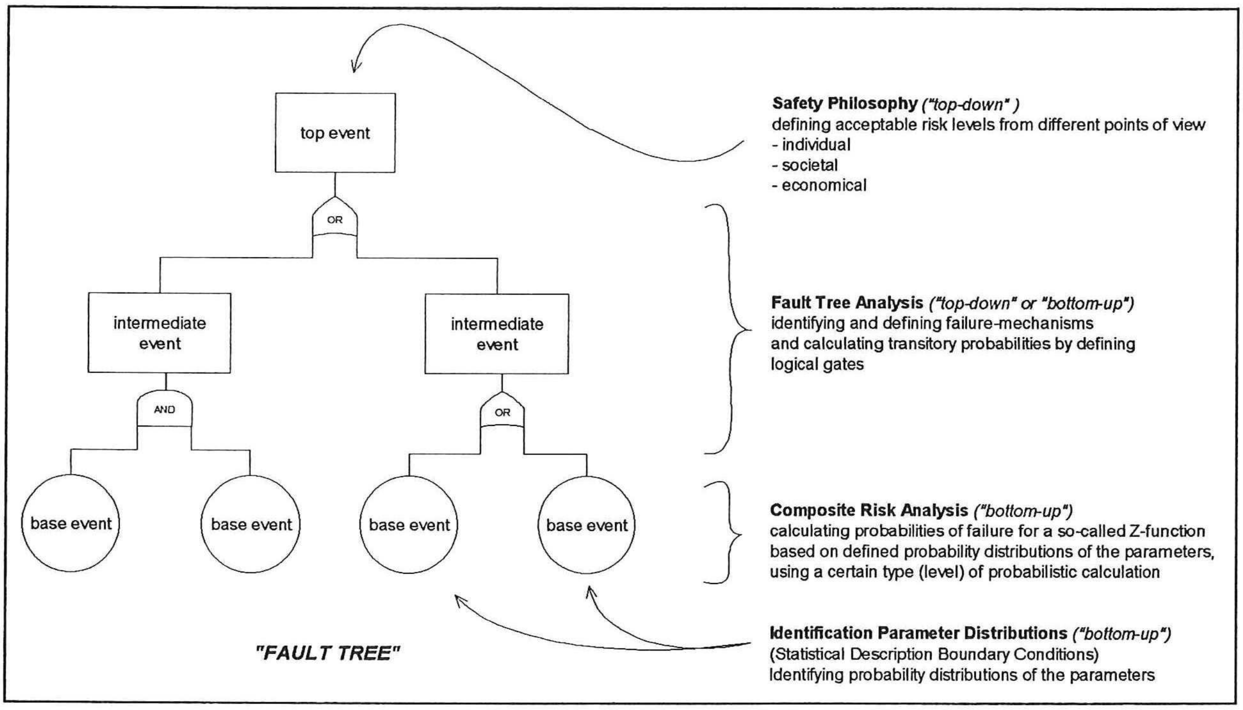
4. The Probabilistic Design Approach

The well-known methodology to account for uncertainty in the design process is called the probabilistic design approach. As opposed to a more classic or 'deterministic' design approach, this approach tries to create a concise and transparent frame-work to design towards structures with a probability of failure that is in agreement with acceptable levels of risk. In this study, risk is defined as:

Risk = Probability of failure x Consequences of failure

A probabilistic design approach is not one single method but can be better described as a family of methods. The main elements of this approach will be discussed here and are illustrated in **Figure 1**.

Figure 1 Main elements of the probabilistic approach



4.1. Defining acceptable probability of failure

One of the first steps in this methodology is to define an acceptable level of risk for the event that the main functions of the structure should fail. This failure is often referred to as 'failure of the top event'. The theory behind the methods to define these acceptable levels is called safety philosophy. In such methods the individual, societal and societal-economical acceptable levels of risk involved with the structure are identified and quantified. In contrast with a deterministic design approach, the possible consequences of the aforementioned failure are taken into account. In a probabilistic approach, a structure with an important function will therefore be constructed safer than one with a less important function. In a deterministic design approach, such a distinction can be made intuitively by the engineer(s), but this will be more subjective and less transparent than in the aforementioned method.

Another positive effect of this method is that it will force all parties involved in the design to agree on definitions of 'a safe level' and 'failure of the structure'. Simply the acceptance of the fact that the structure can fail and the transparency of the trade-offs that have to be made between cost, safety and other considerations, can change the attitude of involved parties. This will probably have a positive effect on the acceptance of the design and more rational reactions of these parties in case of failure of this structure.

4.2. Fault tree analysis and allocation of acceptable probabilities

In order to determine what could lead to failure of a top event, it is necessary to identify the main failure mechanisms. We can distinguish serial and parallel systems of failure and each of these systems can consist of dependent or independent components.

In case of a **serial system**, the system can fail as a result of any of the system's components (like the lights in a Christmas-tree). The probability of failure of the top event if the system components are independent, will be:

$$P_f = 1 - \prod_{i=1}^{n} (1 - P_{f_i}) \approx \sum_{i=1}^{n} P_{f_i} \qquad \text{(for small } P_{f,i}\text{)}$$

If the system components are dependent the probability of failure of the top event will be within a range with upper and lower limit:

$$\max_{i=1}^{n} P_{f_i} \leq P_f \leq \sum_{i=1}^{n} P_{f_i}$$

In case of a **parallel system**, all system components will have to fail before the total system fails. The probability of failure of the top event is, in case of complete independence of the system's components, equal to the product of the probabilities of failure of all components:

$$P_f = \prod_{i=1}^{n} P_{f_i}$$

In case of dependence of the components the probability of failure is in the range:

$$0 < P_f < \min P_{f_i}$$

Once the fault tree is constructed, the acceptable probabilities of failure can be allocated for the intermediate events; the nodes in this fault tree. There are several methods and algorithms to do this allocation. Often the probabilities are evenly distributed over the different branches if the system of failure is parallel. Another option is to optimise economically and allocate high probabilities to 'cheap failures' and low probabilities to expensive ones. Furthermore, it is possible to add so-called 'conditions' that can reduce the acceptable probability of the **ultimate limit state (ULS):** failure, to that of the **service limit state (SLS):** damage. An important effect of this method is that the engineer is forced to look further than the structure itself. The strength and properties of surrounding structures and the environment have to be taken into account as well, because their failure can also lead to the failure of the top event.

In a deterministic design the engineer will use experience to identify the main failure mechanisms and adapt the design according to them. These mechanisms can become very complex, however, and the experience of the engineer might not be relevant for the considered project.

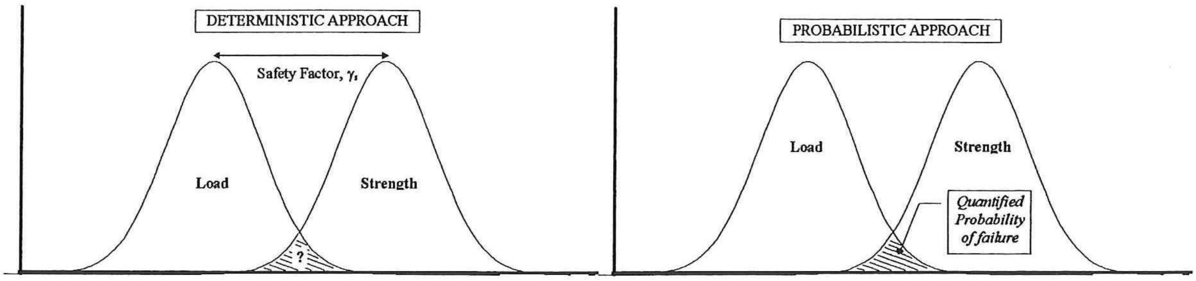
4.3. Probabilistic calculations

To calculate the probability that a certain failure will occur, a probabilistic calculation can be made based on the model for the functional relationship between loads and required strength. Normally the model is represented by a so-called 'Z-function':

$$Z = R - S = Z(X_1, X_2, \ldots X_n)$$

The probability of failure is equivalent to the cumulative probability of all possible events where the load is bigger than the strength, in other words, where $Z < 0$. This is illustrated in Figure 2 by the hatched area under the two curves. In this figure also the deterministic approach is shown where a **safety factor** is usually used to compensate for all kind of uncertainties and create a large enough margin between the load and the strength of the structure (In Figure 2 the safety factor is used on the mean values of strength- and load distribution, this is not necessarily the case in reality).

Figure 2 Deterministic versus Probabilistic approach



This is a very schematic representation of the real situation. The model will probably contain many parameters of which the occurrence or uncertainty can be represented by **probability density functions (PDF)**, of which some of the most often used are shown in BOX 1. This gives the problem a multidimensional aspect. To calculate the probability of failure, several methods are available. These methods can be divided into three levels:

Level III

The exact probability distributions of the parameters are used. One of the methods of calculating with these full distributions is the **Riemann method** where the 'failure space' is determined by full integration of the PDFs. For example, if Z is a function of three parameters, the integration will be:

$$P_f = \iiint_{Z<0} f_{x_1}(x_1) f_{x_2}(x_2) f_{x_3}(x_3) dx_1 dx_2 dx_3$$

Another method is the **Monte Carlo method of sampling** where possible outcomes of the **cumulative distribution function (CDF)** are simulated with random generated numbers (X_u) from a uniform distribution between 0 and 1. With the inverted CDF (F_{Xi}^{-1}), the parameter's values corresponding to these outcomes (X_i) can be deduced.

$$X_i = F_{X_i}^{-1}(X_u)$$

For each simulation run, samples are taken from all parameter distributions and the value of Z is calculated. If these runs are repeated many times, the obtained values for Z can be put in a histogram which approximates the PDF of Z. The probability of failure can be determined by dividing the number of runs which resulted in Z < 0 by the total number of runs.

Level II

In this method all distributions are approximated by normal distributions. If Z is a function of many parameters X_i, all with a normal distribution, then Z can be approximated with a normal distribution by linearisation of the function in a certain point, the so-called design point (X*), this is called the first order-second moment (FOSM) method. If this point is the vector of the mean values of all parameters X this method is called the mean value approach. A more advanced class of level II calculations is the **approximate full distribution approach** (AFDA) where the Z-function is linearised in the point where the probability density of Z is at its maximum. The parameters of the normal distribution of Z can be derived from:

$$\mu_Z = Z(X_1^*, X_2^*, \ldots, X_n^*) + \sum_{i=1}^{n} \frac{\partial Z}{\partial X_i}\left(\mu_{X_i} - X_i^*\right)$$

$$\sigma_Z = \left\{ \sum_{i=1}^{n} \left(\frac{\partial Z}{\partial X_i} \right)^2 \sigma_{X_i}^2 \right\}^{1/2}$$

Level I On this level, partial safety factors are derived for partial loads and strength. This is called **load and resistance factor design** (LRFD). The factors can be determined by combining the two figures in Figure 2: deriving the safety factor according to a defined probability of failure. The most important aspect is that the factors have to be applicable in many situations and therefore the partial loads and strength have to be distinguishable and independent.

4.4. Probability distributions of the parameters

For all parameters involved in the calculations the probability distributions have to be determined. If time series of observations are available, the distributions are obtained by statistical fitting of certain suitable models to these observations and choosing the model that fits best, see BOX 1 for several probability distributions that are frequently used. If no observations are available, these parameters will have to be estimated from information in the literature and theory, and from experience and intuition of the engineer. However, the engineer's experience is not necessarily fully understood or relevant for the considered situations. In this case it is difficult to draw quantitative conclusions based on this.

BOX 1 Frequently used probability distributions

Some of the most important probability functions are given. f(x) is the **probability density function** (PDF), F(x) is the **cumulative distribution function** (CDF). Some functions don't have a 'closed form' of F(x). In this case approximate functions are often available.

Normal distribution and standard normal distribution

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}}\exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right) \quad \text{and} \quad f(z) = \frac{1}{\sqrt{2\pi}}\exp\left(-\frac{z^2}{2}\right)$$

z = standard normal variate = (x-μ)/σ

Approximations for F(x), proposed by Abramowitz and Stegun (1965)

$$F(z) = \frac{1}{2}\left[1 + 0.196854|z| + 0.115194|z|^2 + 0.000344|z|^3 + 0.019527|z|^4\right]^{-4}$$

with F(z) = F(z) for z < 0 and F(z) = 1-F(z) for z ≥ 0 (error less than 2.5e-4)

Lognormal distribution

$$f(x) = \frac{1}{x\,\sigma_y\sqrt{2\pi}}\exp\left(-\frac{1}{2}\left(\frac{\ln(x)-\mu_y}{\sigma_y}\right)^2\right)$$

y = ln (x)

Exponential distribution

$$f(x) = \lambda\exp(-\lambda x) \qquad F(x) = 1-\exp(-\lambda x)$$

Uniform distribution

$$f(x) = \frac{1}{b-a} \qquad F(x) = \frac{x-a}{b-a}$$

a = minimum , b = maximum

5. Discussion probabilistic approach

Usually, the design of a structure will involve both kind of approaches, deterministic and probabilistic, since various fields of study have developed differently towards the use of a probabilistic approach. In hydrological studies, for example, which is often a part of a larger study, statistical analysis of large amounts of available hydrological data has been widely applied for quite some time already. This is part of a probabilistic approach as is shown in **Figure 1**. In fields of study like geotechnical- or geomorphologic analysis, this is less often the case. It is therefore difficult to speak of a pure probabilistic or deterministic design, since it will contain aspects of both.

In theory, a deterministic approach can either under- or overestimate the probability of failure by applying arbitrary safety factors, if we test it against a certain acceptable failure-threshold (e.g. 5%). Intuitively, some people argue that a deterministic approach will more likely over- then underestimate the probability of failure. This is based on the assumption that in time, safety-factors are likely to become more and more safe or, in other words, conservative. This again, is based on the assumption that an individual is generally risk-averse. If a certain safety factor is applied, and structures built with it would fail, policy makers might tend to increase the safety factor, because there was no a-priori information about a probability of failure of the structure and there was no agreement on an acceptable level of risk. In that case the policy makers are acting like a risk-averse individual. In practice, however, there doesn't seem to be proof of consistently more conservative designs if a deterministic approach is used than if a probabilistic approach is used.

From a theoretical point of view, a probabilistic approach is superior to a deterministic approach, because it enables for a consistent way to calculate risk and compare this with other designs. It is a transparent and concise method of accounting for uncertainties in the design, which is applicable in many fields of work. In practical situations, however, not everybody seems to be convinced about the added value of the use of such an approach in every situation. To understand this reluctance to accept a probabilistic approach, we have to look into the nature of the criticism more detailed and into the way probabilistic methods are being applied at this moment. Two of the critical remarks that were encountered during the period of this analysis were:

1. Often there is hardly any information available about the parameters used in the calculations, and even if information is available, it is surrounded with uncertainty. A probabilistic approach forces the designer to adapt a probability distribution based on very limited information, which means quantifying uncertainty without a solid base. This could result in an unjustified confidence in the results of the probabilistic calculations.

2. To make a full and consistent probabilistic analysis, all parts of the analysis and the design would have to be done in a probabilistic way. This means that the probability distributions of all parameters involved in the design would have to be assessed and that every calculation would have to be done probabilistically. The problem is that deriving the uncertainty of certain parameters can be very time-consuming and doing the probabilistic calculations also take more time than deterministic calculations. In practical situations there seems to be a time-constraint.

Like in many decision problems, the goal is to find an optimal solution for several, often conflicting interests. The main conflicting interests in engineering designs are usually the quality or soundness of the design and the cost of the design (including the cost of the design process itself). For example, it is obvious that extensive measurement programs and model tests can improve the quality of the design and decrease the probability that something unforeseen will happen but it is also obvious that the cost of design will be higher than without the measurements and models.

In case of an engineering consultancy, the main cost is the time spent by their consultants. If a contract has been signed for a project granted, the contract sum is fixed from that moment on and the main goal is to optimise time and quality within this constraint. The consultancy will obviously try to 'sell' every tool and method it thinks necessary to obtain a qualitatively sound analysis or design. But there are limits to what can be sold. If applying a certain method would imply spending 25% more time, the budget might prove to be very difficult to sell to the client. If, on the other hand, the consultancy has decided to use the method in a project granted to them, but doesn't succeed in applying it consistently throughout the design because of the time-constraint, there is no actual improvement in the quality of the design.

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SECTION II

Analysis Design-results three projects

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6. Objective section II

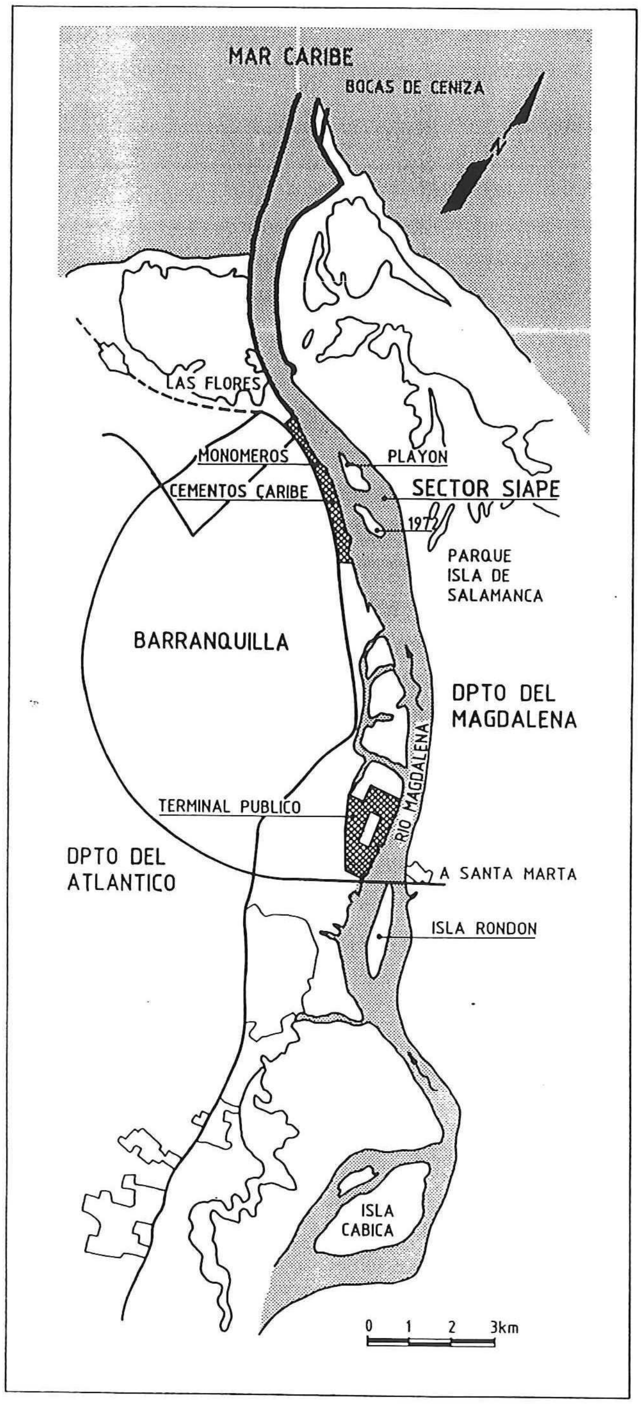
In this section, an analysis is made of three projects in which river training works were designed. The objectives of this section are to study how the probabilistic design method was applied in two of the projects, to study how the uncertainty of the parameters were determined and to quantify the influence of the differences in natural conditions, required safety and used models on the differences in the final designs.

The considered projects are three projects for the design of bank protection works in large rivers, all performed by HASKONING under its own name or under the name of NEDECO.

- Jamuna multipurpose bridge, Bangladesh (NEDECO)
- Meghna river bank protection, Bangladesh (HASKONING)
- Improvement of the access channel to the port of Baranquilla, Colombia (HASKONING)

The projects will be analyzed according to the method presented in the previous section although the chronology of the actual projects might have been different. In the Jamuna and Meghna study, a probabilistic design approach was used and in the Baranquilla study a deterministic design approach. In the paragraph concerning acceptable probability of failure and fault tree analysis the Baranquilla project will therefore not be considered. For the discussion of the calculations and the final results this study will be included again.

Figure 1 Map of Colombia, Rio Magdalena and the project area



7. Introduction of the projects

7.1. Port of Baranquilla (Rio Magdalena) project

The Rio Magdalena is a river which is of vital importance to Colombia. It is a primary fluvial artery into the country and provides an access to the Caribbean Ocean and other inland ports for numerous ports with a large range of products such as petrol, carbon, cement, fertilizer and beer. In recent years, morphological processes have caused deposition of alluvial material in the main channel and along the port facilities at the port of Baranquilla, only a few kilometers from the mouth of the Rio Magdalena, which has made it necessary to carry out maintenance dredging to enable large ships to navigate through the main channel and make full use of the facilities.

The river training works have the objective to direct the main flow of the Magdalena river in westward direction in order to reverse the deposition and prevent future deposition of alluvial sediment in the main channel and on the left bank along the port facilities of Baranquilla.

Design period: 1987 - 1990
Construction period: 1994

The Magdalena river is the largest river in Colombia, about 1,600 km long and drains an area of 255,000 km^2 into the Caribbean Ocean. This corresponds with 23 % of the total size of Colombia. The annual average discharge is 7,500 m^3/s and monthly average discharge varies between 2,000 m^3/s and 10,000 m^3/s.

In Figure 1 a map of the project area is shown. Figure 2 shows a more detailed overview of the port facilities and the proposed river training works. 1 is a rockfill dike guiding the flow towards the left channel, 2 is a sand dike avoiding river flow between the guiding dike and the islands and 3 is illegally dumped land fill and subsequent sedimentation that will be removed.

Figure 2 Access channel port of Baranquilla

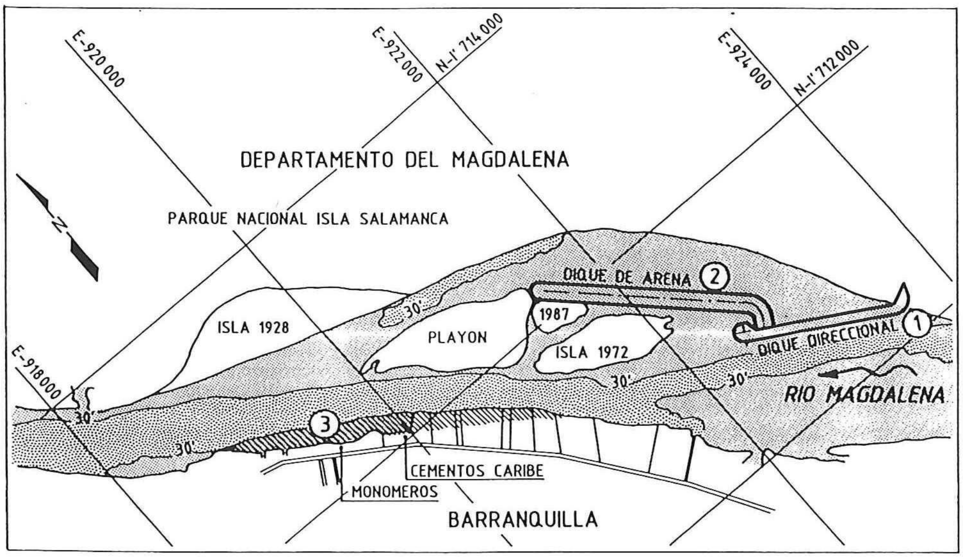
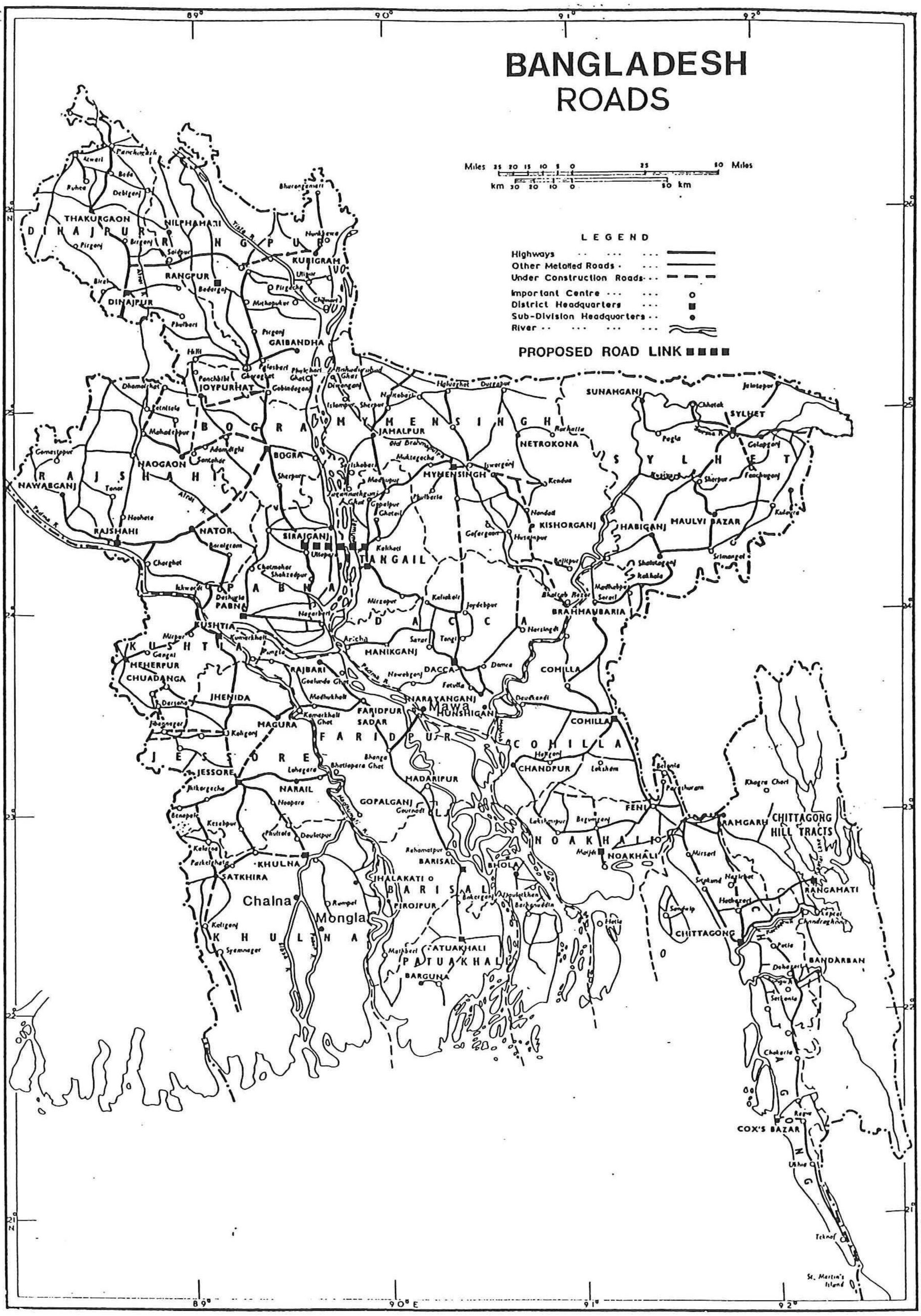


Figure 3 Map of Bangladesh and indication of planned Jamuna bridge



7.2. Jamuna multipurpose bridge project

The Jamuna (the name given in Bangladesh to the Brahmaputra), the Ganges and the Padma constitute a system of rivers which physically divides Bangladesh into East and West Zones, and the West into North and South parts. These physical barriers are seen as an impediment to economic development and social unity. A bridge across the Jamuna would establish the infrastructure for a multipurpose crossing to improve the road and rail transportation networks and provide increased provision of energy to the West. Since 1983, the Government has pursued its interest in a multipurpose crossing of the Jamuna river and by 1988 it figured as one of the Government's highest priorities.

River training works were a vital aspect of the bridge's design. Guide bunds at the site of the bridge and a river bank protection upstream of the bridge at Buapur were designed to protect bridge and river banks against hydraulic loads and the risk of outflanking of the river. See Figure 4 for a detailed lay out of the project area.

Design period:	1986 - 1991
Construction period:	1994 - ?

The Jamuna ranks among the largest rivers in the world. The river is 2,700 km long and drains an area of 580,000 km^2 into the Bay of Bengal, of which less than 10 percent is within the borders of Bangladesh. The average annual discharge is 19,250 m^3/s, the average annual highest flood 65,000 m^3/s and the highest discharge recorded was 90,800 m^3/s. It can be classified as a large braiding sandbed river.

In Figure 3 and Figure 4 ,maps of the project area and the proposed works are shown.

Figure 4 Overview Jamuna bridge and river training works proposed

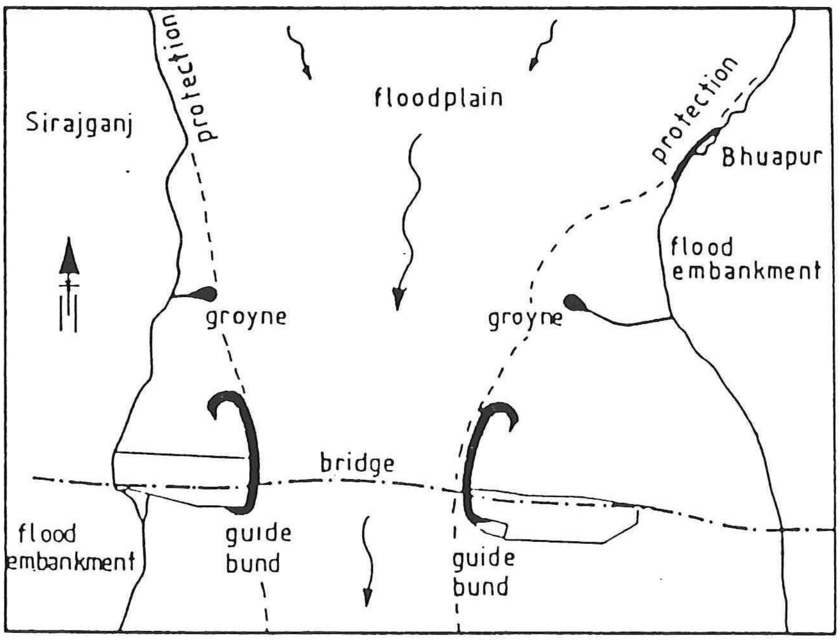
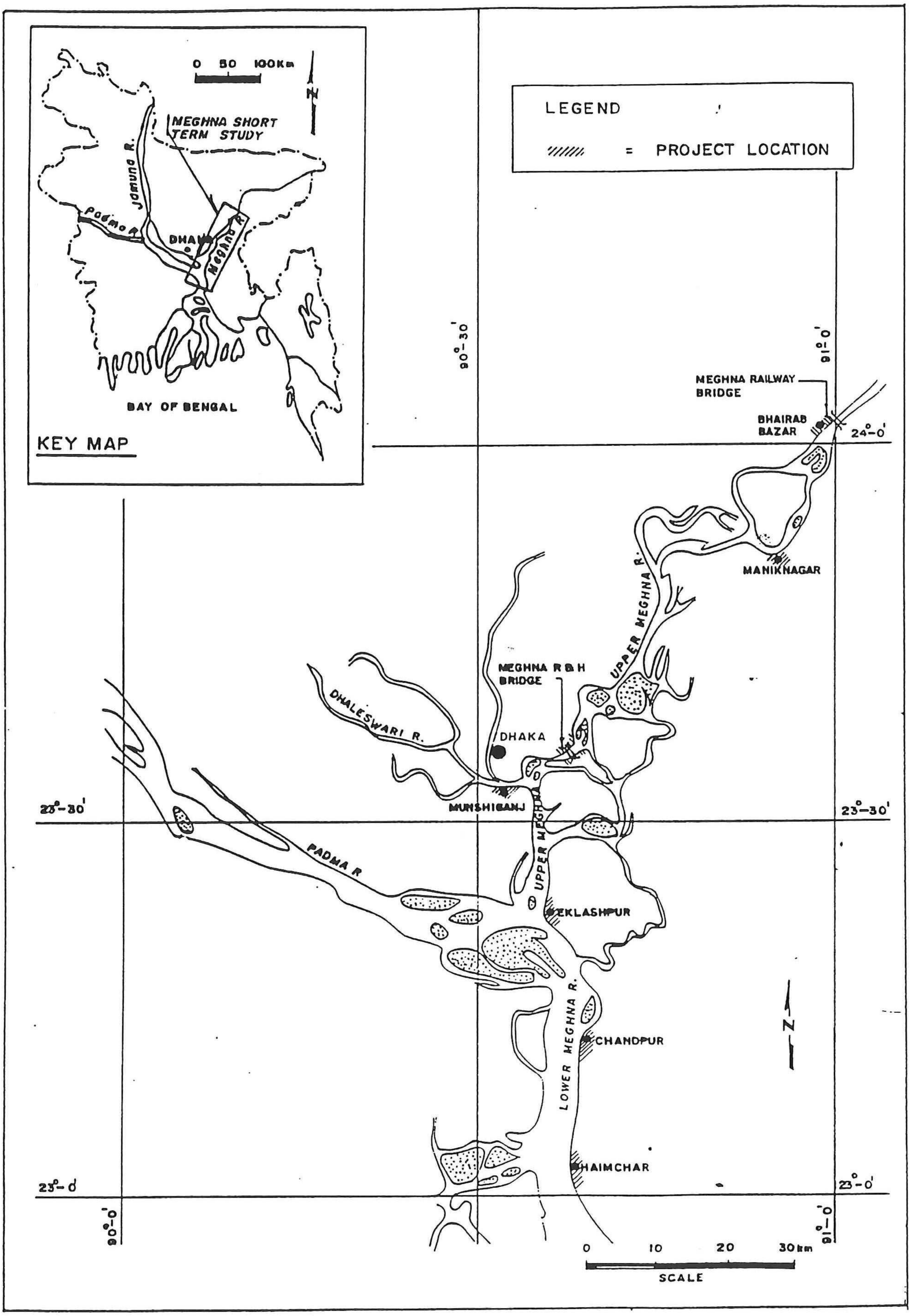


Figure 5 Map of Bangladesh and the project areas



7.3. Meghna river bank protection project

The Meghna, one of Bangladesh's major rivers, flows through the eastern part of Bangladesh and discharges into the Bay of Bengal. Like other rivers in Bangladesh, the Meghna erodes its banks in many points and this erosion has reached an alarming magnitude since the severe floods of 1987 and 1988. Consequently, a number of locations require prompt attention to prevent further damage or even events of a catastrophic nature.

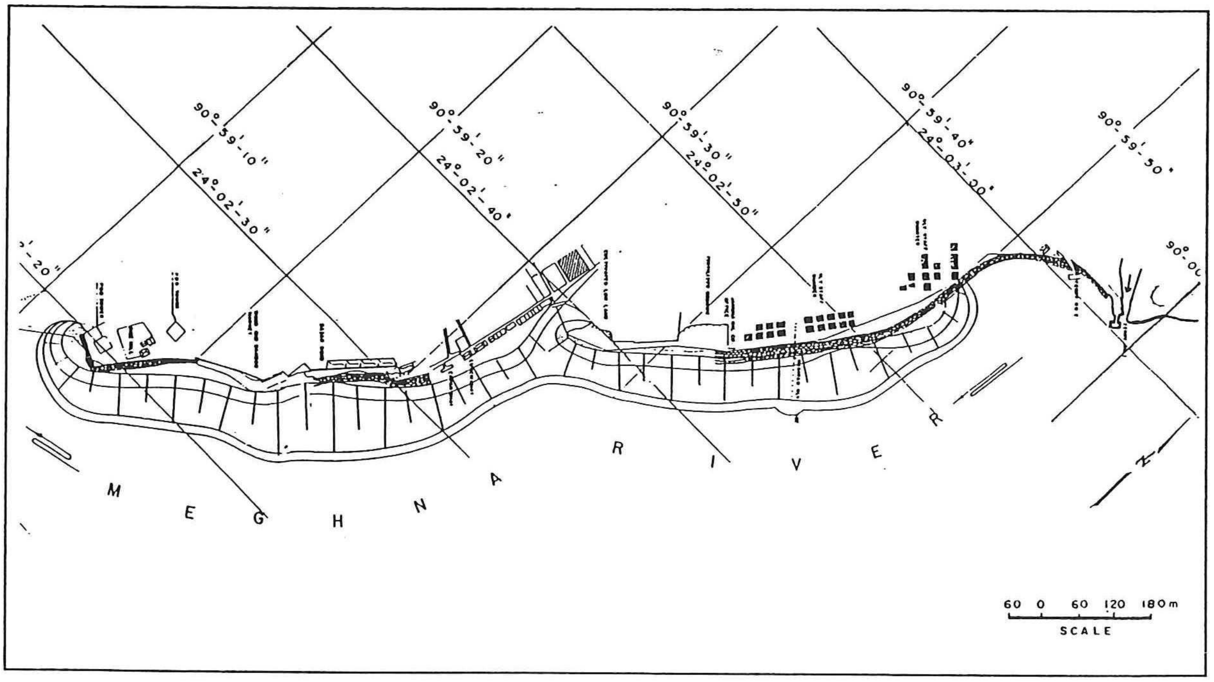
The river training works consist of bank protection works at five sites in the Upper Meghna and three sites in the Lower Meghna.

Design period: 1990 - 1992

The Upper Meghna drains an area of 77,000 km^2 of which about 60 % is in Bangladesh. The annual average discharge is about 4,800 m^3/s and maximum flows can reach up to 19,800 m^3/s.
The Lower Meghna River conveys the water from the Jamuna (Brahmaputra), the Ganges and the Upper Meghna basins. The total catchment area is about 1,637,000 km^2. Maximum flows can be as high as 155,000 m^3/s.

In Figure 5 a map is shown of Bangladesh and a more detailed map of the project area. In Figure 6 the lay-out is shown of the proposed protection at Bhairab Bazar.

Figure 6 Lay-out planned protection Bhairab Bazar

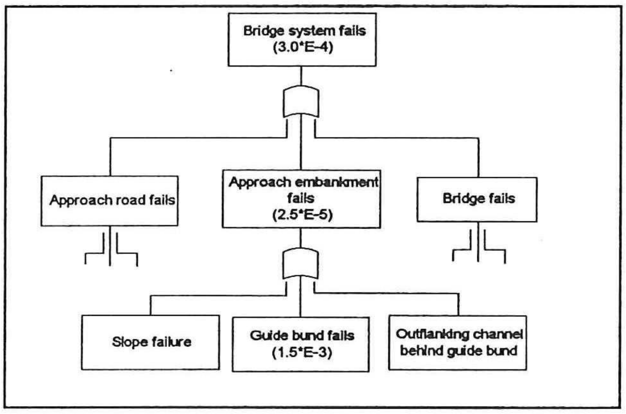


8. Acceptable risk and fault tree analysis

8.1. Acceptable probability of failure

In the Jamuna and Meghna project, a probabilistic approach was used, in the Baranquilla project a deterministic approach. Therefore only the Jamuna and Meghna study are discussed in this paragraph.

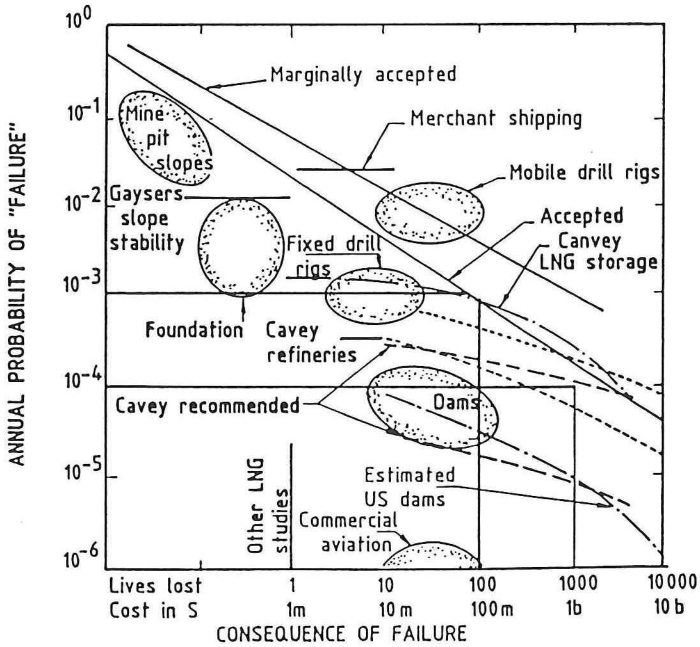
Figure 7 Simplified global fault tree Jamuna project



In the Jamuna project, the guide bunds near the bridge were part of the total bridge system together with the bridge itself, the approach embankments and the approach roads (see Figure 4). The failure of any of these elements that make the bridge fulfill its main purpose leads to failure of this bridge system, which is the top event of the overall fault tree. From a risk analysis, the acceptable probability of failure for the total bridge system was found to be 3.0*E-4. The individual acceptable probability of failure was in the range of 2.0*E-3 ~ 2.0*E-4. Because of economical reasons it was decided not to adopt a level lower than the lower bound of this range. For this analysis, we are interested in the acceptable probability of failure of the top event of the sub - fault tree: failure of the guide bunds. The main function of the guide bunds is the protection of the approach embankments against erosion. Working down the main fault tree, the acceptable probability of the guide bunds was determined to be 1.5*E-3. A simplified global fault tree for the Jamuna project is shown in Figure 7.

For the Meghna project the acceptable probability of failure of bank protections was derived from Figure 8 which shows world-wide observed risk-levels for various engineering structures and activities. The acceptable probability of failure was set at 5.0*E-3. No risk analysis was performed to identify individual, societal or economical acceptable probability of failure.

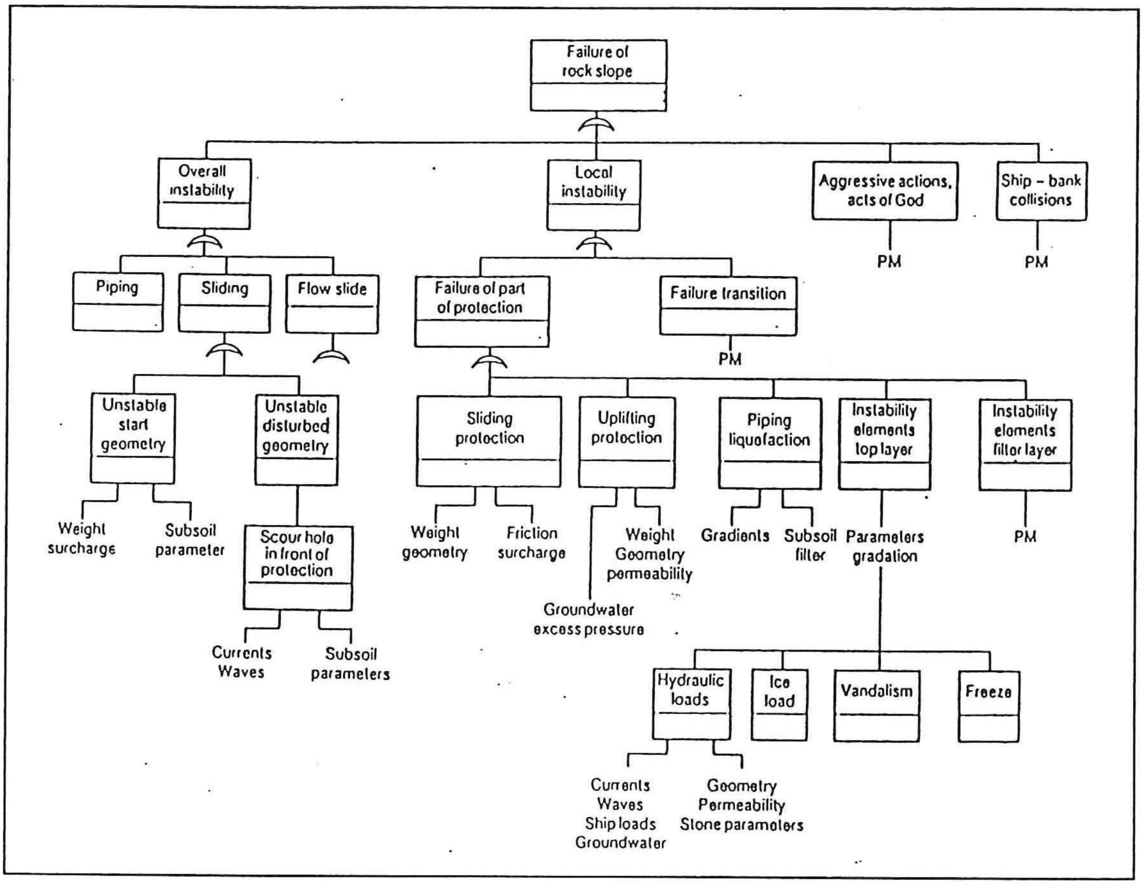
Figure 8 Observed risk for several engineering activities



8.2. Fault tree

The fault trees that were used in both projects are very similar to the fault tree proposed in the CIRIA/CUR 'Manual on the use of rock in coastal and shoreline engineering' which is shown in Figure 9.

Figure 9 General lay-out fault tree bank protections



In this fault tree a distinction is made between overall instability and local instability. The overall instability is mainly due to geotechnical instability and the local instability is caused mainly by currents and waves. The fault trees of the Jamuna and Meghna project are shown in annex A, figure 1 and figure 2.

In these fault trees there is an eminent role for so-called 'conditions' or 'failure transition' (see Figure 9) which were already briefly discussed in section I. These are conditional events for which local failure will lead to an overall failure. For example: damage of the coverlayer of the bank protection will only lead to failure of the complete bank protection if monitoring and maintenance of this coverlayer fails. This is the difference between the 'ultimate limit state' (failure of the protection) and the 'service limit state' (damage of the protection). If the monitoring and maintenance of the structure is reliable, the design of the structure can be lighter and therefore less expensive. For this aspect a difference between the two projects becomes apparent: in the Jamuna project the probability of 'no inspection and maintenance in case of damage' is set at 0.1 because of economic reasons. The structure would become too expensive otherwise. This probability is therefore a kind of required 'maintenance reliability' to obtain this economic optimum. In the Meghna project, a probability of 0.2

appeared to be obtainable. This means that the same failure modes with this condition in both projects have to be designed relatively twice as safe in the Meghna study than in the Jamuna project.
Another example of such a condition is shown in the fault tree of the Jamuna project where some failure mechanisms can only happen if the main channel will shift considerably to one side. The probability that this happens was set at 0.1 which was the result of a morphological analysis.

These conditions are one of the most subjective parts of the fault tree analysis. Some of them can be derived from available data (like the probability of shifting of the main channel from satellite images) but others are based on 'guesstimates', like determining the reliability of maintenance. Firstly it is far from evident how the damage will develop towards failure in time and secondly it is difficult to quantify the probability that damage will indeed be detected, reported and that action will be undertaken to repair it.

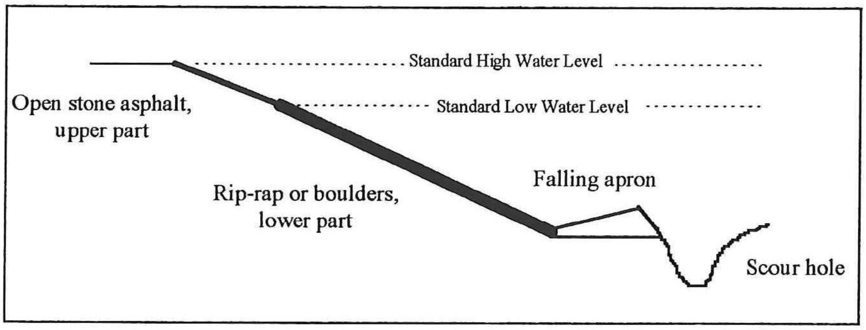
The acceptable probabilities of failure for the different intermediate failure modes were determined through fault tree analysis, like described in section I. The probabilities were in most situations equally allocated to the failure modes. In Table 1, the values derived for the main failure modes are shown.

Table 1 Acceptable probabilities of failure

	Baranquilla	Jamuna	Meghna
Failure total protection	-	1.5E-3	5.0E-3
Damage lower part, current	-	4.0E-2	3.1E-2
Damage upper part, waves	-	4.0E-1	2.5E-2
Damage upper part, current	-	4.0E-2	6.3E-3 (1.5E-1)
Damage due to deep scour	-	2.0E-1	1.2E-1

The elements of the bank protection mentioned in Table 1 are schematically shown in Figure 10. The three main elements in these designs are the upper part, the lower part and the falling apron section. The other layers (backfill, filter, fascine mattress) are not shown in this figure.

Figure 10 Typical cross-section bank protection Jamuna/Meghna project



The probability levels for these two projects are of the same order of magnitude for almost all failure modes, except for "Damage upper part, waves", where the difference is about a factor 10. The reason for this can be found in annex A, figure 1, where the fault tree of the guide bund in the Jamuna study is shown. This fault tree suggests that for erosion of the upper part of the protection to occur, the open stone asphalt has to be damaged by wave attack first. If this remains undetected, probability 0.1, it will be eroded by current attack, with another condition of 0.1. Although this is a correct failure mechanism for open stone asphalt, the second condition is questionable and most probably the result of an error. In the Meghna study, damage by waves and damage by currents are interpreted as two separate failure mechanisms for the upper part. This illustrates that different interpretations or errors concerning the failure mechanisms can easily lead to a factor 10 difference in required safety.

For the Meghna project, the acceptable probability of damage of the upper part due to currents was altered during the report (value between brackets in Table 1). The fault tree, however, was not adjusted (see annex A, figure 2). In order to keep the fault tree consistent, either a condition should be added to this particular failure mode or the fault tree should be recalculated. It appeared that a monitoring and maintenance condition of 0.05 should have been added. Therefore the value between brackets in Table 1 is the correct one.

9. Design calculations

After determining the boundary conditions in hydrological-, morphological- and other partial studies, design calculations were made in all three project to determine the dimensions of the structure. As can be expected, the results of these calculations will be different for the different projects. There are mainly three causes for these differences that can be distinguished:

1. **Natural differences** or physical differences: the natural conditions can be different, for example the velocity in one river is higher than in another. This will be expressed in the (statistical) description of the variables used in the probabilistic calculations.

2. **Safety differences**: structures can be designed with different safety requirements, this will be expressed in the acceptable probabilities of failure determined in the fault trees in the previous paragraph.

3. **Model differences**: using different design formulas can result in differences because they might have been developed by different people, for different conditions and with different accuracy.

In order to compare the three different projects, these differences will have to be quantified. Then it will be possible to compare different probabilistic designs with eachother. For a comparison of deterministic designs, the differences in safety can not be determined directly. If the two other types of differences can be quantified, however, the remaining difference will subsequently be safety difference.

In the three projects, the main design criteria, for which probabilistic calculations were made, are: stability in currents, stability under wave attack and stability of the toe of the protection.

- The stability of the coverlayer elements under current attack appeared to be the main design criteria in all three projects and was therefore the best documented design criteria. Both deterministic and probabilistic calculations were made for this design criteria.

- The stability of the coverlayer elements under wave attack was another main design criteria, although only very few probabilistic calculations were made.

- The calculations of expected scour were done with factors derived from morphological studies and from literature. These morphological studies were not fully documented in some of the reports that were available to the author at the time of the study. It is therefore not possible to make a good analysis of this design criteria.

Although there are more criteria involved in the design of bank protections, most of these were not suitable for probabilistic calculation at the time of these studies. Therefore they will not be discussed here. For example: the geotechnical stability calculations were done with a numerical program using the Bishop method of slip circles to determine safety coefficients. This method did not allow for a level II probabilistic calculation.

BOX 1 Current stability formulas

Izbash (1970):

$$\Delta D = A \, \frac{u^2}{2g}$$

Turbulence

A	=0.3	low turbulence (e.g. normal river flow)
A	=0.7	high turbulence (e.g. return currents due to boats)
A	=1.3	jets (e.g. boat propellers or immediately downstream of control structures)

Shields (1936) -Chézy (1769):

$$\Delta D = \frac{2g}{\Psi_{cr} C^2} \, \frac{u^2}{2g} \quad \text{with} \quad C = 18 \log \left(12h / k_r\right)$$

this equation can be adjusted with a slope factor 1/ K_s and a turbulence factor K_t , the depth factor is embedded in the Chézy formula.

Pilarczyk (1984):

$$\frac{D}{h} = \left[\frac{u}{B_1 \left(\Delta g K_s \Psi_{cr} h \right)^{0.5}} \right]^{2.5}$$

or, rewritten

$$\Delta D = \frac{2}{B_1^2 K_s \Psi_{cr}} \left(\frac{h}{D} \right)^{-0.2} \frac{u^2}{2g}$$

Turbulence

B_1	= 8-10	minor turbulence, uniform flow
B_1	= 7-8	normal turbulence of rivers and channels
B_1	= 5-6	major turbulence, local disturbances, constrictions and outer bends

Pilarczyk (newer version):

$$\Delta D = \frac{0.035 \, \Phi K_t}{K_s \Psi_{cr}} K_h \, \frac{u^2}{2g}$$

Turbulence

K_t	= 1.0	normal turbulence of rivers and channels
K_t	= 1.5	mild bends, below stilling basins
K_t	= 2.0	high turbulence, sharp bends, hydraulic jumps

or $K_t = \left(\frac{1 + 3r}{1.3} \right)^2$ with $r = \frac{\sqrt{\overline{u}^2}}{\overline{u}}$ = relative turbulence intensity [-]

Depth factor

K_h	$=(h/D)^{-0.2}$	undeveloped flow velocity profile
K_h	$=2/(\log[6h/D])^2$	relatively rough units (logarithmic velocity profile)
K_h	$=2/(\log[12h/D])^2$	relatively smooth units (logarithmic velocity profile)

Stability

Φ	= 1.5 - 1.25	exposed edges of loose units
Φ	= 1	exposed edges of block-mats and mattresses
Φ	= 0.75	continuous protection of loose units
Φ	= 0.3 - 0.5	continuous protection of mattress

For all formulas

Δ	$= (\rho_s - \rho_w)/\rho_w$ = relative density of the sediment [-]
D	$= D_n = 0.848 \, D_{50} = (M_{50}/\rho_s)^{1/3}$ = nominal stone diameter [m]
ρ_s	= density of the sediment [kg/m³]
ρ_w	= 1000 = density of water [kg/m³]
ψ_{cr}	= critical Shields parameter [-]
K_s	$= \sqrt{1 - \left(\frac{\sin(\alpha)}{\sin(\phi)} \right)^2}$ = slope factor [-]
g	= 9.81 = gravity acceleration [m/s²]
h	= waterdepth

9.1. Current stability, coverlayer elements

For the lower part of the bank protections rip-rap (rock) was used in all three projects. For this part the stability of the stones in currents is the main requirement. To determine this stability different formulas were used in all three projects:

Baranquilla:	Pilarczyk, version I
Jamuna:	Shields-Chézy, with depth- and slope factor
Meghna:	Pilarczyk, version II

These formulas are quite similar, see comparison of the formulas in BOX 1. The backbone of all formulas is that the effective weight of the required stone, ΔgD, is proportional to the square of the current velocity. Furthermore, the constant to close this relation consists of several factors to account for different kind of reductions or surcharges as a result of waterdepth, bank slope, turbulence etc.

These formulas calculate the D_{50} of the stone-grading for which the stones will be 'stable' under the considered current attack. Complete stability is not possible, however, there will always be some damage of the coverlayer in time. So, actually the probability of a certain damage level should be calculated. Sofar, however, there is no unambiguous definition of damage for current attack available. Nevertheless, these formulas have been used to calculate the damage level referred to in the fault tree, the so-called 'service limit state'.

The design calculations result in a design stone. The size of the calculated design stones are different for all three projects, as was expected. It is possible to determine the possible causes of these differences according to the earlier presented breakdown. In Table 1, the acceptable probability of failure for the coverlayer of rip-rap (lower part) is 4.0*E-2 for the Jamuna project and 3.1*E-2 for the Meghna project. This means that there is hardly a difference in **required** stability of the rip-rap coverlayer. If we assume for the moment that the required stability for the stones in the Baranquilla project will be about the same, it can be concluded that theoretically the differences should be mainly due to natural and model differences. However, it is always possible to include safety factors through the choice of the variables in the deterministic calculations and in the probabilistic designs as well. The probability of failure might be the same for two designs but the intrinsic safety can be higher for one than for the other.

For the purpose of identifying the hidden safety factors and distinguishing between natural and model differences, all the variables in the current stability formulas will be discussed. Of all variables the values applied in the design will be presented. For the Baranquilla project these are deterministic values, for the Meghna report probabilistic and deterministic values and for the Jamuna report probabilistic values and some deterministic values.

In the projects only two probability distribution functions were used, the normal and the Gumbel distribution. are presented with their distribution name and the distribution parameters between brackets. For the normal distribution the notation is NORMAL(μ, σ) with PDF:

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}}\exp\left(-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right)$$

and for the Gumbel distribution this is GUMBEL(u, α) with PDF:

$$f(x) = \frac{1}{\alpha}\exp\left(-\frac{x-u}{\alpha}-\exp\left(-\frac{x-u}{\alpha}\right)\right)$$

9.1.1. Discussion of the variables used in the formulas

Current velocity, u

In order to calculate the stability of the coverlayer elements, the velocity that will exert a force on these elements will have to be determined. This force is known as the shearing stress. To determine this force, the near-bottom velocity is needed. If the depth-averaged velocity is used, a so-called 'depth factor' can be used to compensate for reduced shear stress. This factor will be discussed later. Sometimes the area-averaged velocity is used in cases were it can be assumed that this will not differ much from the depth-averaged velocity in the vertical determining for the stone size.
The current velocity is derived from the hydrological/hydraulic studies in all projects. The results of these studies are summarized in Table 2:

Table 2 Current velocities applied in the projects

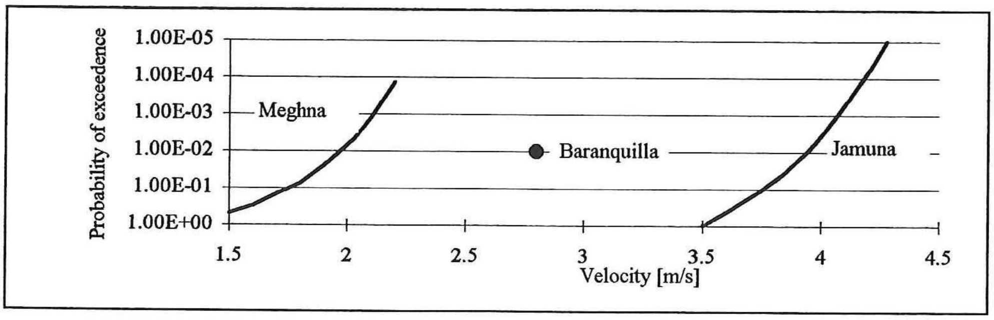
	Baranquilla	Jamuna	Meghna, Bhairab Bazar	
Discharge, Q	12.000	GUMBEL(60862, 6603)	GUMBEL(12971, 1814)	
Stage, H	-	$((Q/1000-3.22)/1.09)^{1/2}+6.04$	$1.0+(Q/1170)^{0.67}$	
Waterdepth, h	20	H + NORMAL(25, 0.10)	H + NORMAL(17, 1.7)	
Roughness, C	85	$0.126\ Q^{0.14} / \sqrt{i}$	NORMAL(70, 7)	
Gradient, i	5.6E-5	7.0 E -5	NORMAL(2E-5, 0.2E-5)	
Velocity, u	2.8	$C \sqrt{(h\ i)}$	$C \sqrt{(h\ i)}$	1.95

The bedlevel in the Jamuna project has an amazingly small standard deviation (1 cm on 25 m) that it could have been set as a constant, which seems unlikely considering the variation in the scour processes. The hydraulic gradient used in this same study is set at a constant value, which is also very unlikely, the standard deviation could at least be 10%. The hydraulic roughness in this study was a function of the discharge but without standard deviation. This means that this would be an exact relation between Q and i, without model uncertainty (see section I), which is very unlikely. However, the absolute variation of C according to this relation is very small (between 68 and 74) so that the model error of it will be hardly significant here.

The standard deviation of the hydraulic roughness in the Meghna project, 10%, is quite large. The 90% confidence interval of this variable is [58, 82] which is, compared with the interval of this variable in the Jamuna river, quite large. A standard deviation of 5% would be more appropriate.

For the Jamuna and the Meghna project, we can calculate the probability that a certain velocity will be exceeded, using the Z-function: $Z = V - C \sqrt{(h\ i)}$ and calculating the probability that $Z<0$ for various V. The velocity used in the Baranquilla project will probably be the 1 in 100 years velocity which is a common design-velocity for these kind of designs. The derived probability functions are shown in Figure 11.

Figure 11 Derived probability functions of the velocity



Turbulence factor, Kt / B1

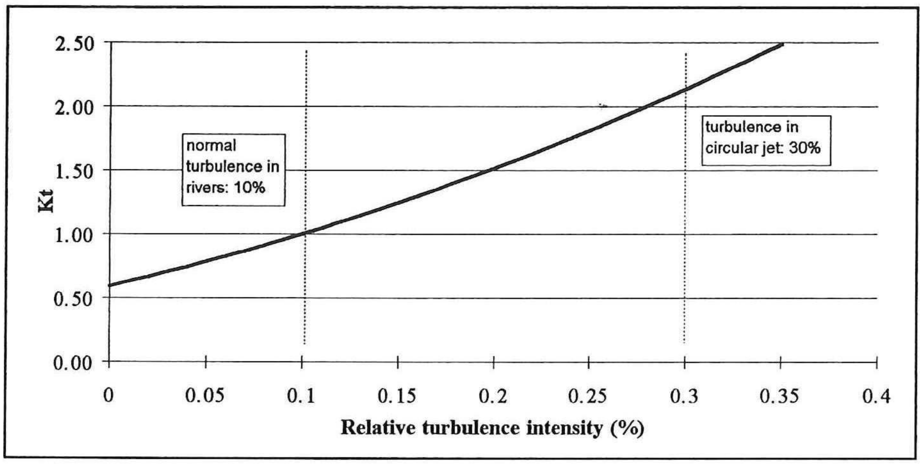
Due to irregular flow patterns, turbulence can cause higher local stresses exerting on the protection elements than the stress determined with the current velocity.
In the Meghna study, the factor K_t was used which is defined as:

$$K_t = \left(\frac{1+3r}{1.3}\right)^2 \quad \text{with} \quad r = \frac{\sqrt{\overline{u'^2}}}{\overline{u}} \quad \text{(r = the relative turbulence intensity)}$$

As a reference, the value of r = 30% (K_T = 2.1) which is found as a maximum value for 'circular jets' can be used as an upper limit. The value for normal turbulence in rivers is about r = 10% (K_T = 1) which can be taken as a minimum value for the preliminary design. For 'plane jets' a value of r = 20% (K_T = 1.5) was found as a maximum value.
In the Meghna project also 'high turbulence' was chosen which resulted in K_t = 1.8 for the deterministic calculation corresponding with a relative turbulence intensity of 25% (higher than for plane jets!). Figure 12 shows the relation between r and K_T.

In the Jamuna project the Shields-Chézy formula was used, but without any factor to account for turbulence. Maybe this turbulence was assumed to be 10% as in normal rivers, but no reference was made for such a choice. In a follow-up phase of the Jamuna study, a value of r = 15% was found from measurements in a physical model.

Figure 12 Turbulence factor Kt, Pilarczyk formula



In the Baranquilla project the tabulated parameter B_1 was used to compensate for turbulence. From a table, 'high turbulence' was chosen which resulted in B_1 = 5~6. In order to compare this turbulence factor with the factors used in the other projects, a factor 1/0.035 was added which resulted in the form shown in Table 3.

Table 3 Turbulence factors applied in the projects

	Baranquilla	Jamuna	Meghna	
Turbulence factor	$2 / (0.035\, B_1^2) = 1.6\sim2.3$	1	Kt = NORMAL(1.5, 0.15)	1.8

In the Baranquilla project and in the Meghna project the engineers have chosen 'high turbulence' in both cases from a table that was available with the formula used. The differences that result from this factor therefore do not come from a perception of higher turbulence in one project than in the other. The differences are therefore **model differences**.

Depth factor, Kh

The depth factor illustrates the dependence of the critical velocity on the waterdepth. For greater depths, a higher permissible velocity has been found.

In the Jamuna project the depth factor in the Shields-Chézy formula is:

$$K_h = \frac{2}{\left(\log[12h/D]\right)^2}$$

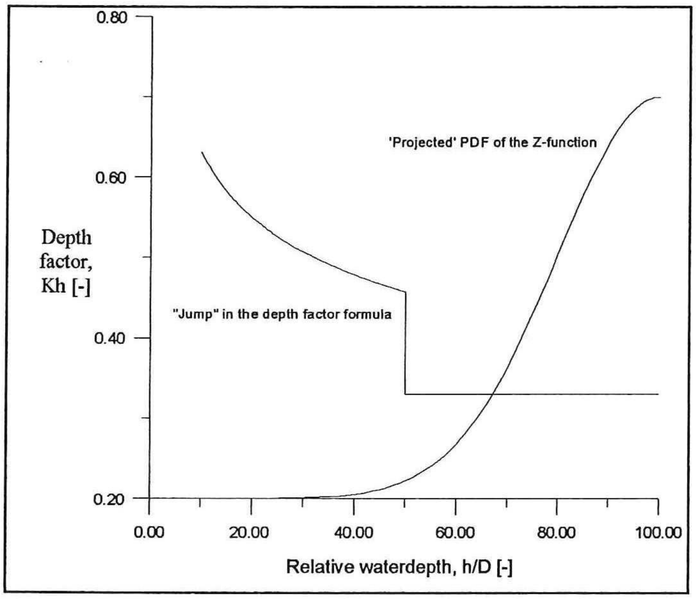
which is limited for a ratio of h/D = 50. Because the waterdepth was for every case more than 50 times D, this factor was defined as a constant at its limit value of 0.33. In this formula the depth factor is obtained by substituting the Chézy roughness with the Nikuradse expression for rough or smooth protection units. The depth factor defined for undeveloped flow is not an available option in this model.

In the Baranquilla study the depth factor was hidden in the original formula of Pilarczyk. After rewriting this formula see BOX 1, the depth factor appears to be

$$K_h = \left(\frac{h}{D}\right)^{-0.2}$$ with no limit value defined in this project.

The other definitions for rough and smooth units are not available in this model. Because the design depth was about 20m and the design stone about 0.4m, h/D is 50 with a corresponding depth factor of 0.46.

Figure 13 'Jump' in the Z-function



In the Meghna study, the same depth factor was used in the calculations and the limit value was programmed at 0.33 for h/D > 50 although the actual upper value is 0.46. This creates a jump in the Z-function, see Figure 13. This discontinuity can cause numerical problems, the program might not be able to converge to the design point if the design point is near this 'jump'. However, since this was not the case in the Meghna study, no such problems were encountered. This jump can, however, cause great differences in calculation results.

Figure 3 in annex A shows some of the results of probabilistic calculations in the Meghna study. The difference in the probability of failure of the two calculations is a factor 100. This difference is entirely caused by the depth factor, which is the only factor that has changed between the two calculations. For the upper part, the relative depth is about 30 (on one side of the jump) and for the lower part it is about 70 (on the other side of the jump). The depth-averaged velocity is in both cases the same, only the near bank-depth is different. Even if this is considered to be physically correct for such a great difference in depths (14 meters), a factor 100 is clearly too much of a difference.

Figure 14 shows the depth factors that can be applied in the Pilarczyk formula and their limitations.

Figure 14 Depth factor Kh, Pilarczyk formula for current stability

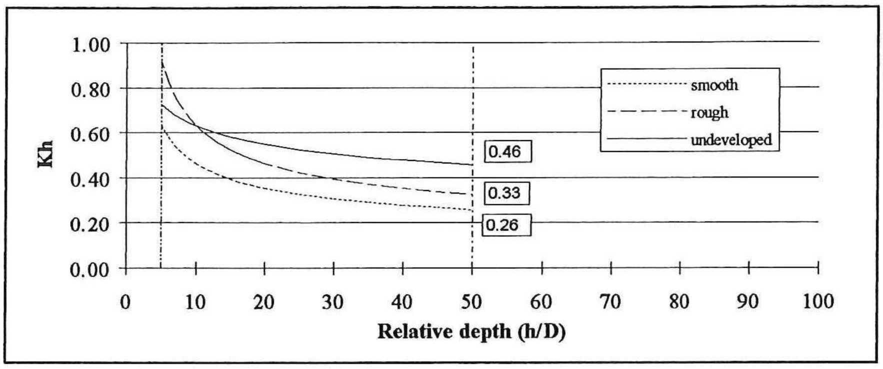


Table 4 Depth factor applied in the projects

	Baranquilla	Jamuna	Meghna
Depth factor, K_h	0.46	0.33	0.33

We can conclude that the differences between the depth factors are hardly the result of a choice made by the designers based on underlying physical differences between the projects, because the designer's options were in all cases limited by the definitions of the models used. Therefore these are **model differences.**

Slope factor, Ks

The slope factor accounts for the reduced resistance against current, of elements on a slope because of a smaller effective weight perpendicular to the slope. This factor can be deduced as:

$$K_s = \sqrt{1 - \left(\frac{\sin(\alpha)}{\sin(\phi)}\right)^2}$$

The slope of the bank protection and the angle of internal friction are the two variables in this factor. The values used in the projects are presented in Table 5. In the Baranquilla project, the chosen slope is 1:4 and in the other two projects this was 1:3.5. Figure 15 shows the effect of the slope factor for various angles of repose and side slopes. The effect on the slope factor is rather small for slopes of 1:3.5 or 1:4. If the slopes become steeper, the effect can become considerable.

Figure 15 Slope factor K_s

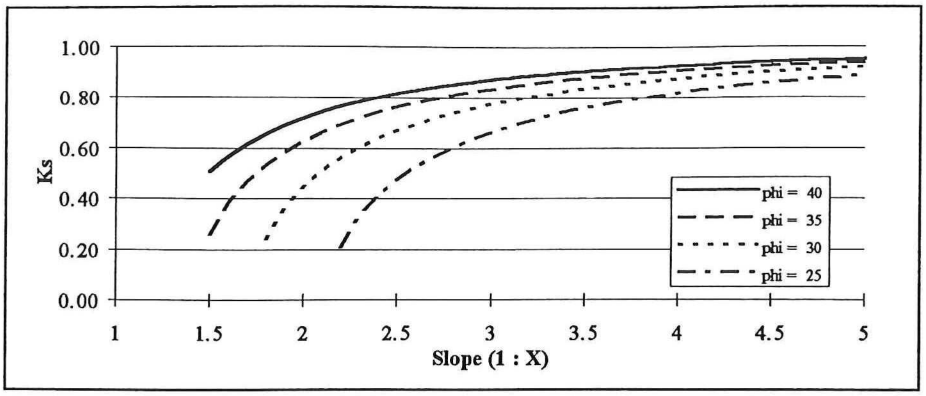


Table 5 Side slope of bank protections applied in the project

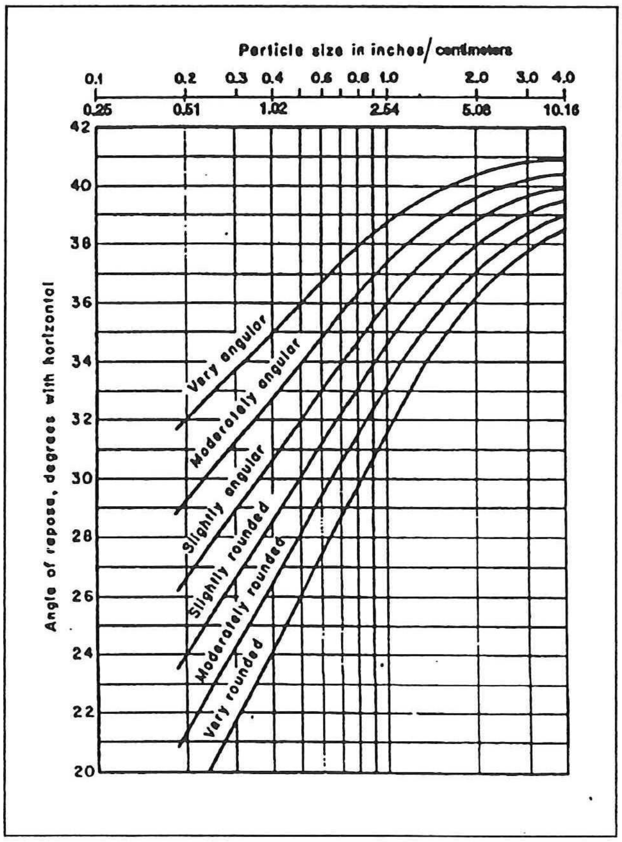
	Baranquilla	Jamuna	Meghna	
Slope of coverlayer	14°	NORMAL(16°,3°)	NORMAL(16°,1.6°)	16°
Angle of friction	35°	NORMAL(37°, 3°)	NORMAL(35°, 3.5°)	35°
Ks	0.91	0.89	0.88	0.88

The obtainable accuracy of the slope, α, of the coverlayer depends on the method of construction. If the coverlayer is placed by hand on a dry slope at low water a higher accuracy can be obtained than in the case where the stones are dumped on the slope from a boat at high water. This is the result of a complex interaction of many factors such as the method of dumping the stones and the system to position the ship and monitor the dumped layer.

But even the accuracy of the slope prepared on a dry slope will decrease over time when waves and currents can displace a certain percentage of the elements. Furthermore, unequal settlement of the subsoil will contribute to this inaccuracy.

The angle of internal friction, ϕ, for loose sands and gravel is usually in the range 30 - 45°. Figure 16 suggests that for D_n > 10 cm, this friction angle is about 40°. For a granular skeleton of rock stones, high contact forces can occur and cause the rock to break. This process is also influenced by weathering of the stone in time which can soften these contact points.

Figure 16 Friction angle for non-cohesive materials (USBR)



Stability factor, ϕ

Only the latest version of the Pilarczyk formula has a stability factor. This is a tabulated factor which has to compensate for the difference in stability of different protection types, like CC-blocks or cellular mattresses. It can also compensate for the difference between a continuous protection or loose edges of protections which are obviously less stable because of the lack of interlocking.

Table 6 Stability factor applied in the projects

	Baranquilla	Jamuna	Meghna	
Stability factor	-	-	NORMAL(1, 0.1)	1

Shields parameter, ψ_{cr}

This parameter indicates a critical value for the dimensionless shear-stress:

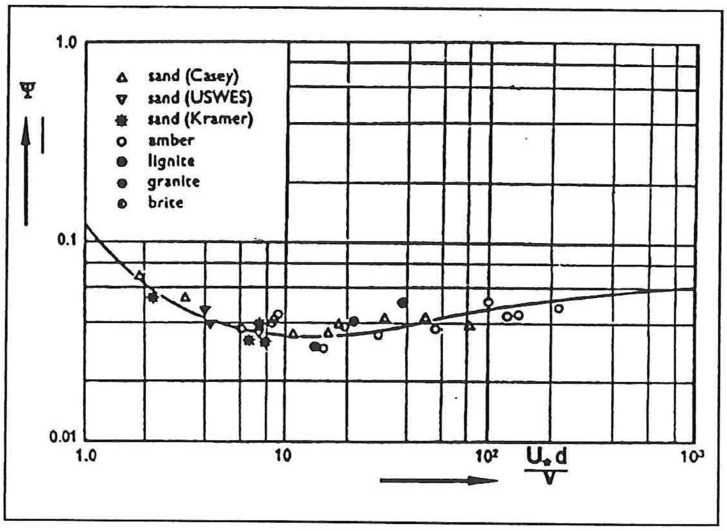
$$\tau_c / (\rho_s - \rho_w) g D_n$$

In 1936, Shields derived a relation between this parameter and a grain related Reynolds-number:

$$U^* D_n / \nu$$

This relation is shown in Figure 17.

Figure 17 Shields diagram



In the Baranquilla project, the threshold of no movement was defined at ψ = 0.03. In the other projects higher values were used for the average value in the probabilistic calculations with a standard deviation of about 10%. In the Jamuna project it was also mentioned that for deterministic calculations a value of 0.03 should be used.

Table 7 Critical shear stresses applied in the project

	Baranquilla	Jamuna		Meghna	
Shields stress	0.03	NORMAL(0.04, 0.004)	0.03	NORMAL(0.035, 0.004)	0.035

Comparing the deterministic values, there is no different state of nature in the three projects. In the handbook used in the Meghna project the threshold of no movement for rock was 0.035, in the other projects there were handbooks that said 0.03. The differences are therefore **model differences**.

Constant

In all three formulas, a constant 'closes' the models. For the formula used in the Baranquilla project a constant was extracted from the turbulence factor, see turbulence factor, to make comparison more sensible. Arbitrarily, the value of 0.035 was chosen using the Pilarczyk formula used in the Meghna project as reference. The differences between these constants are clearly **model differences**.

Table 8 Constant value applied in the projects

	Baranquilla	Jamuna	Meghna
Constant	0.035	$g/18^2 = 0.030$	0.035

Relative density, Δ

The relative density is defined as

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w}$$

In the Baranquilla project, the density of the type of stones available appeared to be 2400 kg/m^3 , which is considered as a low quality of stone. In the other two projects this density was 2600 kg/m^3 .

> *"In general, dealing with one type of rock in a quarry, the 90% exceedence value is not more than 100 kg/m3 less than the average density."* [1]

If a good estimate of the average density is available, an estimate for the standard deviation would therefore be about 100/1.28 = 78 kg/m^3. The factor 1.28 is the standard normal variable of a one-sided, 90% probability interval of the normal distribution. If no good estimate is available, a value of 100 kg/m^3 can be proposed as more appropriate.

The density of water was set at a constant value of 1000 kg/m^3 for the Baranquilla and Jamuna project. In the Meghna project this variable was a probabilistic variable with a coefficient of variation (CV) of 10%. The density of water varies with the temperature and the salinity of the water according to the formula:

$$\rho_w = 1000 + 1.455\ CL - 0.0065\ (Te - 4 + 0.4\ CL)^2$$

$$CL = (S - 0.03) / 1.805$$

Te	=	temperature (°C)
CL	=	chlorinity (‰)
S	=	salinity, total dissolved salt (‰ ratio)

For fresh water conditions and temperatures varying from 0 °C to 20 °C, the change in water density is negligible, about 1‰. For variable salinity, however, the influence is slightly greater. For example, the density of sea water is 1025 kg/m^3. In a saline environment, if the mean salinity is known, a coefficient of variation of 1% will be enough. A coefficient of 10% is clearly too much.

[1] CIRIA/CUR, Manuual on the use of rock in coastal and shoreline engineering

Table 9 Stone density applied in the projects

	Baranquilla	Jamuna	Meghna	
Water density	1000	1000	NORMAL(1000, 100)	1000
Stone density	2400	NORMAL(2600, 25)	NORMAL(2600, 100)	2600
Relative density	1.4	$(\rho_s - \rho_w)/\rho_w$	$(\rho_s - \rho_w)/\rho_w$	1.6

Stone diameter, D

In all projects, the stone diameter calculated was interpreted as the D_n or $D_{n,50}$ which is the nominal diameter. This diameter is equivalent to the sides of a cube with mass M_{50}

$$D_{n,50} = \left(\frac{M_{50}}{\rho_s}\right)^{1/3}$$

with M_{50} (or W_{50}) corresponding to the 50% mass of the stones (statistical median).
It is also often defined as

$$D_{n,50} = 0.85\ D_{50}$$

with D_{50} corresponding to the 50% size of the stones (statistical median). According to the CUR/CIRIA manual for the use of rock:

> *"Gradings of rock fulfilling the class limit specification may be expected to have standard deviations in $D_{n,50}$ varying from 1% for heavy gradings to 7% for wide gradings."*

If there is serious doubt about whether the gradings will indeed fulfill the class limit specifications, a coefficient of variation of about 10% as is used in the Jamuna and Meghna project, can be proposed to be appropriate. If there is no serious doubt about the gradings, a coefficient of variation of about 5% (in the range 1% - 7%) would be more appropriate.

In Table 10 the required stone diameter calculated in the studies is shown. For Baranquilla the values for respectively B=6 and B=5 are given.

Table 10 Stone diameter required for the projects

	Baranquilla	Jamuna	Meghna	
Stone diameter, $D_{n,50}$	0.27~ 0.39	NORMAL(0.18, 0.018)	NORMAL(0.09, 0.009)	0.09

9.1.2. Discussion of the design results

The results of the deterministic and probabilistic calculations will be compared with reference calculations made by the author based on the outcome of the discussion of the variables in the previous paragraph.

For this comparison, the Shields-Chézy formula with slope and turbulence factor was used as the reference formula:

$$\Delta D_n = \frac{K_h K_T}{K_s} \frac{2g/18^2}{\Psi_{cr}} \frac{u^2}{2g}$$

with depth factor $K_h = 2 / (\log(12\,h / k_r))^2$
turbulence factor $K_T = ((1 + 3r)/1.3)^2$
slope factor $K_s = (1 - (\sin(\alpha)/\sin(\phi))^2)^{1/2}$

This formula contains only measurable variables and only one model factor, the critical shields stress. The position of the fitted line and the scatter of the points around this line in Figure 17 are the distribution parameters of this model factor.

By splitting up a model factor in partial model factors, as is done in other equations, the model becomes less suitable for probabilistic calculation because in that case, it is not clear if the partial model factors should all be seen as factors with a separate probability distribution. If yes, how can their distributions be determined?

A problem with non-measurable, tabulated parameters is that the user might not be able to relate them to the real situation. For example: what is exactly "high turbulence"? Could it be K_T = 1.5, 1.8 or rather 2? In this case it would be more helpful to tabulate the measurable variable r, of which K_T is derived, and give various practical example of values found in real situations. For example: the estimation of channel resistance coefficients with examples of real situations is an example of such a method which is elaborated in French[1986].

Deterministic design

For the deterministic reference design the following values were applied:

- The critical shields stress in the reference formula was set at 0.03, the value most often used for "no movement".
- The depth factor was used for hydraulically rough conditions ($K_r = 2D_n$) limited at 0.33 for $h/D_n > 50$.
- For all three projects the turbulence was set at r = 25% corresponding with $K_T = 1.8$. This is a very conservative estimate for it is a 'free jet'-like turbulence, but is was chosen as an average of the values applied in the projects.
- The variables identified as natural conditions are the same for the applied and the reference calculation.

In Table 11 the variables used in the applied and the reference calculation are listed. The calculated stones are also shown in Figure 18.

Table 11 Comparison of the deterministic designs

	BARANQUILLA		JAMUNA		MEGHNA	
	applied	reference	applied	reference	applied	reference
u (1:100 yrs)	2.8	2.8	3.9	3.9	1.95	1.95
K_T	1.6 ~ 2.3	1.8	1	1.8	1.8	1.8
K_h	0.46	0.33	0.33	0.33	0.33	0.33
K_s	0.91	0.91	0.89	0.89	0.88	0.88
Ψ_{cr}	0.03	0.03	0.03	0.03	0.035	0.03
Δ	1.4	1.4	1.6	1.6	1.6	1.6
Factor	0.035	0.03	0.03	0.03	0.035	0.03
Dn	0.27 ~ 0.39	0.19	0.18	0.32	0.08	0.08
Model/Safety difference	+ 42% ~ + 105%		- 44%		0%	

The results in Table 11 indicate that the difference of the applied deterministic calculation with the deterministic reference calculation can be as much as a factor 2. The factors that cause the differences are indicated by the hatched cells in the table. Because the natural differences have been eliminated between the reference and applied calculations the difference can only be attributed to model- and safety differences.

For a deterministic design the safety can not be quantified directly. From the table it shows that most of the difference is caused by the depth factor and the turbulence factor. In the previous discussion of these two variables, these differences were identified as mainly model differences. Therefore we can conclude that the differences in deterministic designs are almost completely caused by differences in the definition of the used models.

Probabilistic design

For the probabilistic reference design the following values were used:

- The critical shields stress was defined as NORMAL(0.04, 0.004).
- The relative turbulence intensity, r, was defined as NORMAL(20%, 2%). As in the deterministic design, this is a quite conservative value (a mean value of a 'plane jet'-like turbulence) which is not backed by measurements. It is taken as an average of the values applied in the projects.
- The depth factor was in all cases set at a constant value of 0.33, assuming that the design depth would be more than 50 times D_n.
- All other factors were left unaltered except for minor changes in standard deviations of some variables (see discussion variables)

The complete results of applied and reference probabilistic calculations are shown in figures 4 and 5 in annex A. In
Table 12 and Figure 18, the summarized results are shown. Because the design calculations in the projects were not calculated for the acceptable probability of failure, but accepted a lower probability of failure, two reference calculations were performed: one with the applied probability of failure and another with the acceptable probability of failure. In this way a breakdown can be made between safety-differences and model differences.

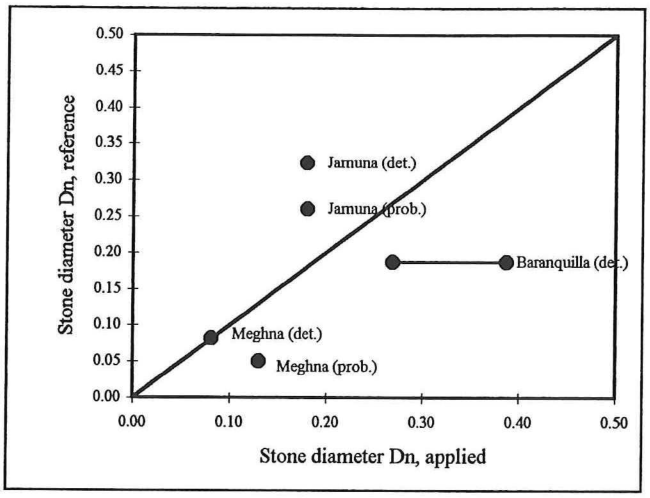
Table 12 Comparison of the probabilistic designs

	Jamuna ($P_{f,acc}$ = 4.0E-2)		**Meghna** ($P_{f,acc}$ = 3.1E-2)	
	D_n	$P_{failure}$	D_n	$P_{failure}$
A. Design applied, $P_{f,\ applied}$	0.18	8.6E-3	0.13	2.2E-4
B. Reference Design, $P_{f,applied}$	0.29	5.6E-3	0.07	3.8E-4
C. Reference Design, $P_{f,acc}$	0.26	4.2E-2	0.05	3.1E-2
Total difference (1-A/C)	- 31%		+ 160%	
Safety difference (1-B/C)	+ 12%		+ 40%	
Model difference (A/C-B/C)	- 43%		+ 120%	

The results show that after eliminating the contribution of the safety difference, the model differences result in a design-stone that is more than a factor 2 bigger or smaller than it is supposed to be. The main sources of these discrepancies are the differences in the mean values of the turbulence and the critical shields parameter adopted. These differences were previously discussed and identified as being model differences. As with the deterministic designs, we can conclude that the differences in probabilistic designs are almost completely caused by differences in the definition of the used models.

All design results presented in Table 11 and
Table 12 are shown in Figure 18, where the stone diameters determined in the projects are plotted against the results of the reference calculations. The design-stones above the middle line are under-designed compared to the reference calculations and the ones under this line are over-designed.

Figure 18 Deterministic and probabilistic calculations vs. reference calculations



Compared to the reference calculations, the deterministic and the probabilistic design-stones in the Jamuna project are under-designed. This is mainly caused by the turbulence factor.

Compared to the reference calculations, the probabilistic design-stone in the Meghna report is over-designed. This was caused by the practical design consideration that the minimal layer-thickness on a fascine mattress should be 0.30m and that there should be at least two layers of stone in this layer. Although this led the designer to choose a minimal D_{50} of 0.15m, it is very well possible to obtain the same thickness with smaller stones.

Compared to the reference calculations, the deterministic design-stone range (for $B_l = 5$ and 6) in the Baranquilla project is over-designed. This is mainly due to the applied turbulence factor for 'high turbulence'. In the Baranquilla project the following standard stone gradings were used:

$$W = 20 \text{ kg} \quad \Rightarrow \quad D_{n,50} = 0.20 \sim 0.23$$
$$W = 80 \text{ kg} \quad \Rightarrow \quad D_{n,50} = 0.32 \sim 0.37$$

According to the design stone, a stone grading of W = 80 kg would only be sufficient for part of this range (for high turbulence of $B_l = 5$ even a larger stone should be applied), according to the reference calculations, a stone-grading of W = 20 kg would be sufficient.

BOX 2 Wave stability formulas

Rule of the thumb (-) *(for open stone asphalt)*	$$D_{layer} = C\,H_s$$ C = coefficient depending on subsoil = 1/6 for open stone asphalt on filter fabric
Hudson (1953)	$$\Delta D = \left(K_D \cot(\alpha)\right)^{-\frac{1}{3}} H_s$$ K_D = 3-4 natural rock K_D = 3.5 plunging waves (later revised to 2) K_D = 4 surging waves K_D = 8-10 artificial elements
Van der Meer (1988)	$$\Delta D = \left(6.2 P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}\right)^{-1} H_s$$ for plunging waves, $\xi_m < 2.5\sim3$ $$\Delta D = \left(P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{P} \sqrt{\cot(\alpha)}\right)^{-1} H_s$$ for surging waves, $\xi_m > 3$ $\xi_m = \frac{\tan(\alpha)}{\sqrt{H/L_0}}$ = Irribarren number [-] P = permeability factor [-] N = number of waves [-] S_d = damage level [-]
Pilarczyk (1990)	$$\Delta D = \left(\Phi_s \Psi_u \cos(\alpha)\, \xi_m^{-b}\right)^{-1} H_s$$ Φ_s = stability factor [-] Ψ_u = stability upgrading factor [-] b = interaction exponent [-], $0.5 \le b \le 1.0$
For all formulas	α = slope angle [°] Δ = $(\rho_s - \rho_w)/\rho_w$ = relative density of the sediment [-] D = $D_n = 0.848\, D_{50} = (M_{50}/\rho_s)^{1/3}$ = nominal stone diameter [m] (= diameter of a cube with same volume as considered non-spherical stone) ρ_s = density of the sediment [kg/m^3] ρ_w = 1000 = density of water [kg/m^3] H_s = significant wave height [m]

9.2. Wave stability

For the upper part of the protection in the Jamuna and Meghna project, open stone asphalt was used, a gap-graded mixture of mastic and stone. In the Baranquilla project a breakwater type of stone was applied. For this part of the structure, stability under wave attack is the main requirement. The formulas used to determine this were the following:

Baranquilla:	Hudson (rip-rap, breakwater type)
Jamuna:	Rule of the thumb (open stone asphalt)
Meghna:	Rule of the thumb (open stone asphalt)

As for the current stability formulas, these formulas are all very similar, see **BOX 2**. The backbone of these formulas is the proportionality of ΔD with H_s, the significant wave height. This is the wave height defined as the average of the highest 1/3 of all waves or in other words, exceeded by 13,5% of all waves.

9.2.1. Discussion of the variables used in the formulas

Significant wave height, Hs

The significant wave height, H_s was in all projects theoretically derived from the wind speed, u_w, with the Bretschneider formulas for wave forecasting:

$$H_s = H' (u_w / g)^2 \qquad \text{and} \qquad T_m = T' (u_w / g)$$

where H' and T' are the dimensionless wave height and wave period, defined as:

$$H' = 0.283 \tanh (0.0125 F'^{\,0.42}) \qquad \text{and} \qquad T' = 7.54 \tanh (0.077 F'^{\,0.25})$$

where F' is the dimensionless fetch length defined as:

$$F' = g F / u_w^2$$

in which F is the fetch length.

In the Jamuna project wave heights could be derived from a distribution fitted to the 15 minute wind speed (m/s): $V_{mean,\,15min} = 15 + 2.25 \ln (T)$
in which T is the return period in years (no fitting results available). This can be written as an exponential distribution with parameters u = 15 and $\alpha = 2.25$

$$F(x) = 1 - \frac{1}{T} = 1 - EXP\left(-\frac{V-u}{\alpha}\right)$$

The short notation will be EXPON(u, α). Although a statistical description of the wave height is available through this distribution, a 1 in 10 years wave height was derived and used for further design calculation. The 1 in 10 years wind velocity is 20 m/s and the corresponding wave height is 0.93 m.

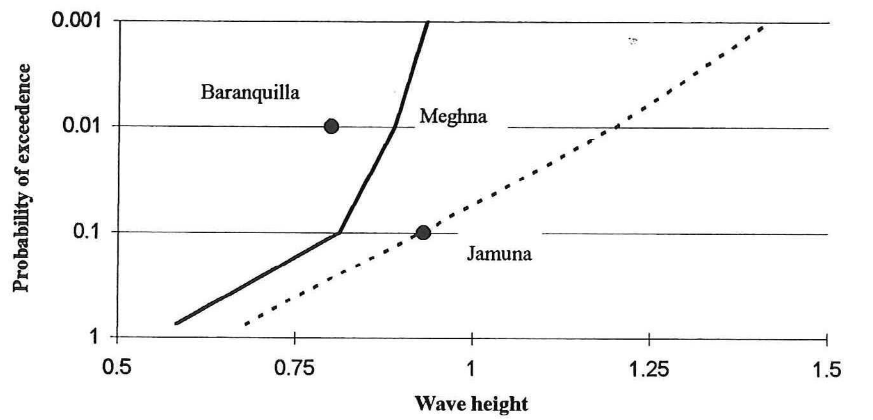
In the Meghna project a Gumbel distribution was fitted to the wave heights that were derived from the wind speed (no fitting results available). In the Baranquilla project the waves caused by ships appeared to be higher than those induced by the wind. A significant wave height of 0.8 m was applied. There was no frequency analysis included for the occurrence of these ship-waves. For the moment it will be assumed that this is a 1 in 100 years wave height, which is a very common design return period for these kind of designs.

The values for all variables involved are summarized in Table 13 and the derived probability of exceedence of the wave height is shown in Figure 19.

Table 13 Wave heights applied in the project

	Baranquilla	Jamuna		Meghna (Bhairab Bazar)	
Wind velocity (m/s)	16.7	EXPON(15, 2.25)	25.4	25.6	
Fetch length (m)	2000	4000		2000	
Waterdepth (m)	7	10		25	
Wave period (sec)	-	-		3.51	
Wave height, wind	0.65	0.93		GUMBEL(0.72, 0.11)	0.98
Wave height, ships	0.80	-		-	-

Figure 19 Probability of exceedence of the wave height



Proportionality factor

In the Baranquilla project no probabilistic calculations were made for this design criteria. In the Jamuna project a semi-probabilistic calculation was done in which a single wave height was determined for the acceptable probability of failure (translated into the return period) but without a variation on the model factor. In the Meghna project deterministic and probabilistic calculations were made with the 'rule of the thumb'.

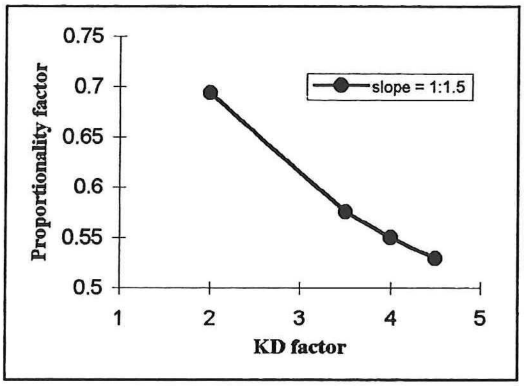
Table 14 Proportionality factor applied

	Baranquilla	Jamuna	Meghna (Bhairab Bazar)	
slope protection, α	1:1.5	(1:3.5)	(1:3.5)	
K_D	3.5	-	-	
proportionality factor	0.53	1/6 = 0.167	NORMAL(0.15, 0.015)	0.167

For the 'rule of the thumb' - factor, 1/6, no information was available for the standard deviation (the model error of the 'rule of the thumb'). The probabilistic calculation in the Meghna project uses an estimate for this model error (10%) which is based on a 'guesstimate'.

In PIANC, "Guidelines for design and construction of flexible revetments..", reference is made to the Shore Protection Manual which states that the value of K_D is defined as 3.5 for breaking (shallow-water) waves, but was recently revised downward to 2.0. It is also stated, however that this manual is not very clear in its definitions hereabout. This revision would result in 25% larger stones, see Figure 20. The K_D for non-breaking (deep-water) waves is said to be equal to 4.0. In the CUR manual, however, the K_D factor is said to have an average value of 4.5 with a coefficient of variation of 18% for $K_D^{1/3}$. This illustrates that although K_D is a well-tabulated factor, there is no consensus among the various handbooks of what these values should be exactly.

Figure 20 Proportionality factor vs. KD factor



Thickness of the layer / stone diameter

The thickness of the open stone asphalt layer in the Jamuna and Meghna project and the design stone in the Baranquilla project that were applied, are shown in Table 15. The density of the open stone asphalt was not used in the calculations, but it is shown as illustration.

Table 15 Applied stone diameter / thickness of the layer

	Baranquilla	Jamuna	Meghna	
Stone density, ρ	2400	(2150)	(2150)	
D (diameter/thickness)	0.33	0.15	NORMAL(0.15, 0.015)	0.15

9.2.2. Discussion of the design results

In this paragraph only the deterministic design results will be discussed. The reason for this is that the only probabilistic design calculation was done with the 'rule of the thumb' - formula which is not suitable for probabilistic calculations, because no information about the model error is available.

The only formula applicable for open stone asphalt, besides the rule of the thumb, is the formula of Pilarczyk with tabulated variables, which is recommended in PIANC, "Guidelines for design and construction ... ".

$$\Delta D = \left(\Phi_s \Psi_u \cos(\alpha)\, \xi_m^{-b}\right)^{-1} H_s$$

Φ_s = stability factor [-]
Ψ_u = stability upgrading factor [-]
b = interaction exponent [-], $0.5 \leq b \leq 1.0$

With this formula, reference calculations were made for the deterministic designs of the Jamuna and the Meghna project. In figure 6 and 7 in annex A, the results of these calculations are shown. The D_n in these figures should be interpreted as the layer thickness. A summary of these results is shown in Table 16.

As a reference for the Baranquilla project the Van der Meer formulas have been used, see BOX 2. The main improvements of these formulas, compared to the Hudson formula, is that account is given to the wave period (T), the storm duration (N) and the structure's porosity (P) and that a clear definition of damage is provided. This makes these formulas much more suitable for probabilistic calculations than the Hudson formula, because the probability that a certain damage will be exceeded in a certain lifetime can be calculated. According to the CIRIA/CUR manual for the use of rock, the coefficients in these formulas also have a much smaller standard deviation than K_D , which means that the model error is smaller than the one of the Hudson formula:

> *"The reliability of van der Meer's formula can be expressed by giving the coefficients 6.2 and 1.0 in equations (...) and (...) a normal distribution with a certain standard deviation. The coefficient 6.2 can be described by a standard deviation of 0.8 (variation coefficient 6.5%) and the coefficient 1.0 by a standard deviation of 0.08 (8%). These values are significantly lower than that for the Hudson formula at 18% for Kd^1/3 (with mean Kd of 4.5). With these standard deviations it is simple to include 90% or other confidence bands."*
> [CIRIA/CUR, p266]

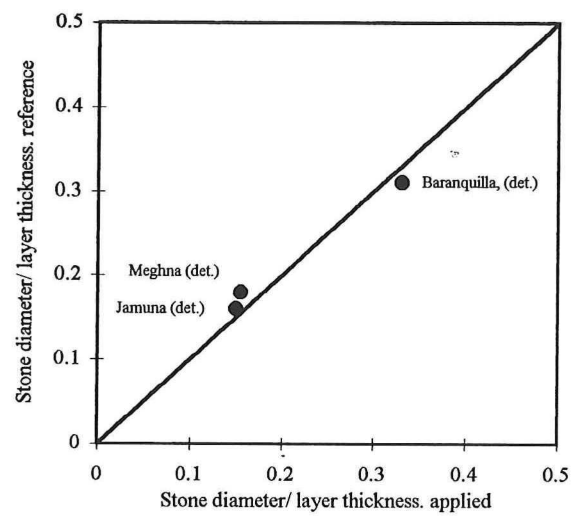
In this analysis these formulas will only be used for a deterministic calculation. In annex A, figure 8, the results of this deterministic calculation are shown. A summary of these results is shown in Table 16.

Table 16 Deterministic calculations layer thickness, applied and reference

	BARANQUILLA ($P_{f,acc}$ = ?)		JAMUNA ($P_{f,acc}$ = 4.0E-1)		MEGHNA ($P_{f,acc}$ = 2.5E-2)	
	applied	reference	applied	reference	applied	reference
Hs	0.8	0.8	0.93	0.82	0.98	0.86
Φs	-	3.48	-	2.71	-	2.71
Ψ_u	-	-	-	2	-	2
b	-	-	-	0.67 .	-	0.67
ξ_m	-	2.62	-	1.29	-	1.26
α	33.7°	33.7°	-	16°	-	16°
K_D	3.5	-	-	-	-	-
Δ	1.4	1.4	-	1.15	-	1.15
'rule of thumb' - factor	-	-	1/6	-	1/6	-
D	0.33	0.31	0.15	0.16	0.15	0.17
Total difference (appl/ref-1)	+ 6%		- 6 %		- 12%	

The wave height of the reference calculations are derived from the probability distributions discussed earlier, see Table 13, and the acceptable probabilities of failure determined in the fault tree, see Table 1.

Figure 21 Deterministic calculations vs. reference calculations



The thickness of the open stone asphalt layer in the Jamuna and the Meghna project deviate only 6 % to 12 % from the reference calculations, despite the differences in the model applied. The design stone of the Baranquilla project is also in accordance with the reference calculation. This indicates that the differences in the models used (rule of thumb against Pilarczyk and Hudson against Van der Meer) do not cause significant discrepancies between the (semi-)deterministic designs in the projects analyzed.

10. Cost-effect of the observed differences

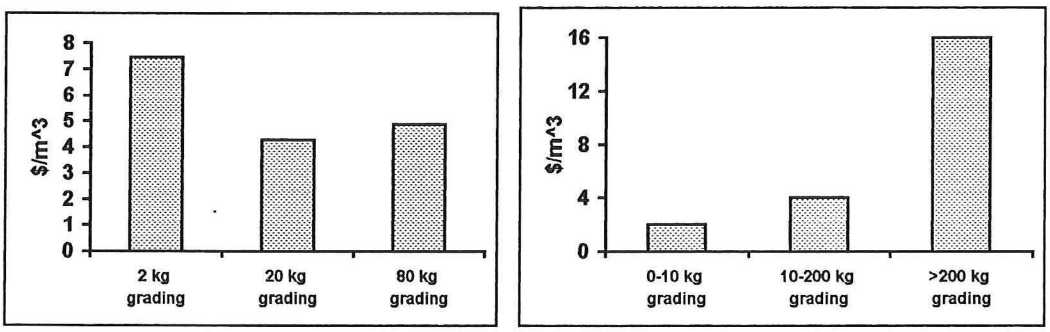
In the previous paragraphs it was found that the differences in design stone can be up to a factor two, mainly as a result of model differences. An important question is whether these differences result in significant differences in the cost of the structure. This will determine the incentive to reduce the model differences. The effect on the cost of the structure will largely depend on the cost-curve of the stone sizes of the stone quarries where the stones will be obtained.

Stone quarries often work with standard stone-gradings with more or less fixed prices for each grading determined by production cost and market demand. If the stone demand of the project considered is small compared to the total output of the quarry, these prices are not likely to change much because of this demand. In this case the differences observed in the design stone can be directly translated into a cost difference. For the Baranquilla project, the reference calculations resulted in a required stone grading of W=20 kg, priced at \$4.25 / m^3 (see Figure 22). The calculations performed in the project resulted in a required stone grading of W=80 kg, costing \$4.90 / m^3 (if the maximum turbulence factor, B_l = 5, is used even a larger stone grading is needed). Here, the model differences are directly responsible for an extra cost of \$ 0.65 / m^3.

If the standard stone-gradings are used that are presented in the article "Quarry based design of rock structures", this price difference can be even bigger if the design stone is around 0.30m or 0.40m, see Figure 22. These gradings correspond to the following $D_{n,50}$:

W = 0 - 10 kg	⇒	$D_{n,50}$ = < 0.30	⇒	\$ 2.0 / m^3
W = 10 - 200 kg	⇒	$D_{n,50}$ = 0.30 ~ 0.40	⇒	\$ 4.0 / m^3
W = > 200 kg	⇒	$D_{n,50}$ = > 0.40	⇒	\$ 16.0 / m^3

Figure 22 Cost of stone gradings Baranquilla & Cost of stone gradings article[2]



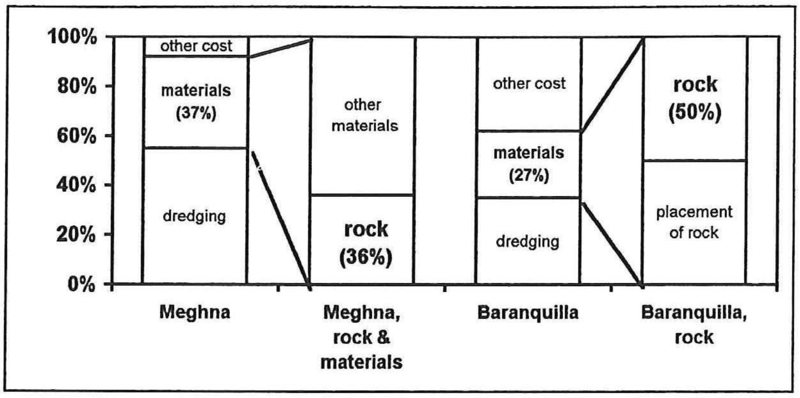
In some projects, however, the quantity of stones demanded is very large compared to the output of the quarry. It is also possible that the stones will have to be obtained from more than one quarry. For the projects in Bangladesh the stones will even have to be obtained from quarries in India, some of them especially developed for the considered projects. In these cases the stone demand will largely determine the price of these stones and price-differences between gradings will very likely be more gradual and smaller.

The effect on the total cost of the structure can be determined by making a break-down of the total project cost into material cost and non-material cost and breaking the material cost down into cost of

[2] Leeuwestein, et al., Quarry based design of rock structures, Rip-rap workshop, Colorado, 1993.

stone and cost of other materials[3]. The results of this break-down for the Meghna and the Baranquilla project are shown in Figure 23. It should be noted that these are projected costs and not actual costs.

Figure 23 Breakdown of the total constructional project cost



Note: costs that are added to a project's subtotal as a certain percentage (e.g. 10% provision), are not included in this breakdown. It is assumed that these percentages are equally allocated over the different activities.

From Figure 23 we can see that the cost of stones account for 37% x 36% = 13.3% of the total project cost in the Meghna project and for 27% x 50% = 13.5% of the total cost for the Baranquilla project. For the Meghna project the dredging cost was quite important because the advanced protection alternative was considered. In the Baranquilla project the mobilisation cost and supervising cost were considerable. The cost of dredging the channel on the left side were not taken into account in this analysis because they are not a part of the bank protection.

The effect of the design differences on the total project cost can simply be determined by:

$$\text{cost difference (in \% of total cost)} = \frac{p_2 - p_1}{p_1} * \frac{\text{cost of stones}}{\text{Totalcost}}$$

where p_1 and p_2 are the prices for the stones of two different designs. For the Baranquilla project this would result in:

$$\frac{4.90 - 4.25}{4.25} * 13.3\% = 2.0\%$$

The total project cost increased with 2% in the Baranquilla project because of model differences. As mentioned before, this influence on the project cost can be even more considerable if the price differences between stone-gradings are bigger.

There are ways, however, to reduce large differences in price between rock-gradings. If the structure consists of several elements requiring different stone sizes, the article "Quarry based design of rock structures" (Leeuwestein et al.) shows that a supply-based design of rock structures, instead of a demand-based design, can considerably reduce excess cost of rock production in the quarry. This excess cost is the cost of the stone produced by the quarry but not actually used in the structure. A supply-based design will, however, necessitate an alliance between design engineer and quarry manager in order to mutually benefit from these potential cost-savings.

[3] The absolute cost is confidential and therefore not mentioned in this analysis

11. Conclusions

In only two of the projects a probabilistic design approach was used: the Meghna and the Jamuna project. The Baranquilla project was designed deterministically. The acceptable probability of failure of the total structure was for the Jamuna and the Meghna project of the same order of magnitude, about 1.5*E-3 ~ 5.0*E-3.

In the fault tree analysis, most of the failure modes were defined at the 'service limit state' (damage). If monitoring and maintenance fails, this will eventually lead to the 'ultimate limit state' (failure). These kind of 'conditions' are one of the most subjective elements of the fault tree because there is often little knowledge and information about the transition of 'damage' to 'failure'. It is important for a design engineer to realize the quantitative aspects of a fault tree. A different interpretation of failure mechanisms can easily lead to acceptable probabilities of failure that differ a factor 10 from eachother.

In order to compare the final design results of different projects, the design calculations have to be split into the three elements that cause these differences: **natural differences**, **safety differences** and **model differences**. The natural differences are enclosed in the boundary conditions, the safety differences can only be directly quantified by comparing probabilistic calculations and the model differences can be derived through comparison of the formulas and the model factors used. By comparing all calculations with reference calculations, all based on the same model and the boundary conditions for each project, these differences can be identified.

Some of the design considerations were calculated probabilistically in the Jamuna and the Meghna project: stability of the coverlayer under **current attack** and **wave attack** and the **expected scour** in front of the structures. Only the first two aspects could be studied in this section because there was insufficient information of the third aspect available to distinguish between natural and model differences.

Certain formulas frequently used in the design of river training works are not very suitable for probabilistic analysis. Models used in probabilistic design calculations should calculate the probability of the events identified in the fault tree, mostly 'damage' or 'failure' of one of the structure's elements. The formulas for current stability, for example, do not calculate damage to the structure over a certain period of time, but a stability limit. The Van der Meer formulas for wave stability are examples of suitable models for probabilistic calculation, because they include a clear definition of damage.

Certain variables in the formulas used in design calculations are variables that are not directly measurable, often with tabulated values for the various conditions. Although this can assist the design engineer in a preliminary deterministic design, it is not very suitable for probabilistic calculations. The main problem is the assessment of the uncertainty of such variables because the designer can not refer to measurements made in the ongoing or previous projects. The model uncertainty should preferably be represented as the standard deviation of a single model factor that 'closes' the equation.

One of the main difficulties concerning probabilistic calculations is the assessment of the variable's uncertainty if no measurements are available. In the projects analyzed, the designers tended to assign a standard deviation of about 10% of the variable's value in these cases. For some of the variables this is clearly too much (water density, ρ_w) or too small (hydraulic gradient, i_h). If no measurements are available, other sources can prove to contain useful information for the estimation of probability distribution parameters. Examples of these kind of sources are: reports of similar projects, handbooks articles etc.

Probabilistic calculations were performed with the Advanced Full Distribution Approach (AFDA), a level II probabilistic calculation method. Level III methods were not used. In some cases, the Z-function were programmed discontinuous but this didn't result in numerical instability because the jumps in the Z-functions were not near the design point.

The analysis in this section indicated that most of the differences between the design calculations in the projects and the reference calculations were caused by model differences. These differences can be up to a factor two. This means that a certain engineer can design elements of a coverlayer that are twice as big or as small as another engineer using a different design handbook, simply because of the different formulas used or the tabulated or prescribed variables in this formula. The factors that contribute most to these differences are the turbulence factor (K_t), the depth factor (K_h) and the critical Shields stress (Ψ_{cr}).

The effect of these differences in design stone on the cost of the project can be determined by using the cost-curve of the stone size. This curve depends greatly on the existing quarries and the relative importance of the stone demand of the project for these quarries. With the break-down of the project into cost of the rock and other costs the relative difference on the total project cost of the design stone differences can be determined. In order to avoid great price differences between stone sizes and to optimize total structure cost, a quarry-based design might prove to be useful.

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SECTION III

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Section III: Analysis of the Design Process

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12. Introduction Section III

Studying the design results of a project can not give a full insight in how is dealt with all the uncertainties encountered during the analysis- and design process of this project. For this reason the author has been involved in the first phase of an ongoing project in Bangkok, Thailand:
the Mekong River Bank Erosion Study (MRBES).
In this section, unlike in the previous section where final results were discussed, the emphasis is on the **design process**. Some results will be given, however, to illustrate the applied methods and give the reader a comprehensive picture of this project. It should be kept in mind, however, that the project was still in progress at the time of this thesis-study and therefore the results are not final.

One of the most important observation during the study of the analysis- and design process of this project was that in many cases the uncertainties encountered were left unanalyzed because of reasons like time-constraint, inadequate 'design-tools' or lack of agreement on how to communicate uncertainty. Most of these problems are related to project-organizational and software-technical aspects rather than problems with the probabilistic approach itself. Because time and information are two of the most vital aspects for any consultancy, these are very relevant aspects to study.

13. Objective Section III

The objective of this section is to discuss the main probabilistic elements in the design process, identify the main problems and obstructions that hinder a full and consistent probabilistic approach and develop methods and instruments to alleviate these obstructions. The MRBES will be used as a example case for these methods and instruments. Some of the main elements of the design-process that will be discussed in this section are the following:

- ⇒ determining uncertainty of observed parameters and derived parameters
- ⇒ identifying and managing the 'flow' of the parameters through the design process
- ⇒ communication of uncertainty throughout the design process (e.g. from one partial study to another)
- ⇒ deterministic and probabilistic calculations

These elements are illustrated in Figure 1.

Figure 1 Main elements of the Design process for probabilistic approach

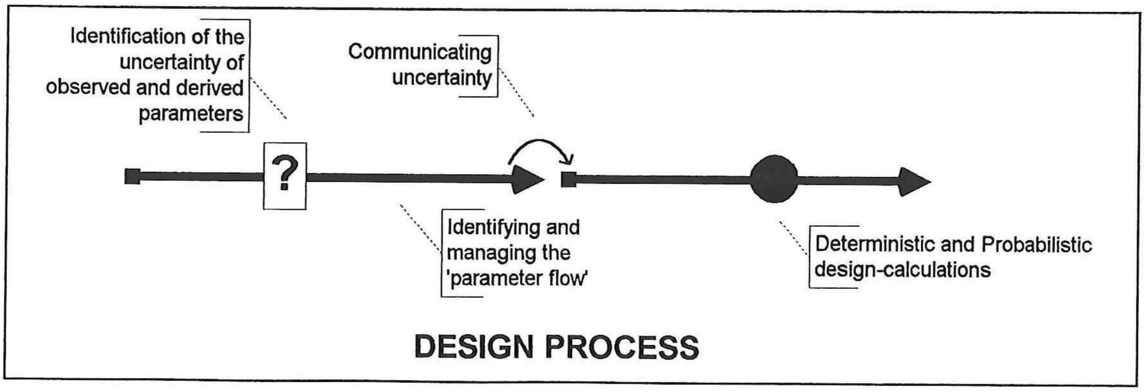
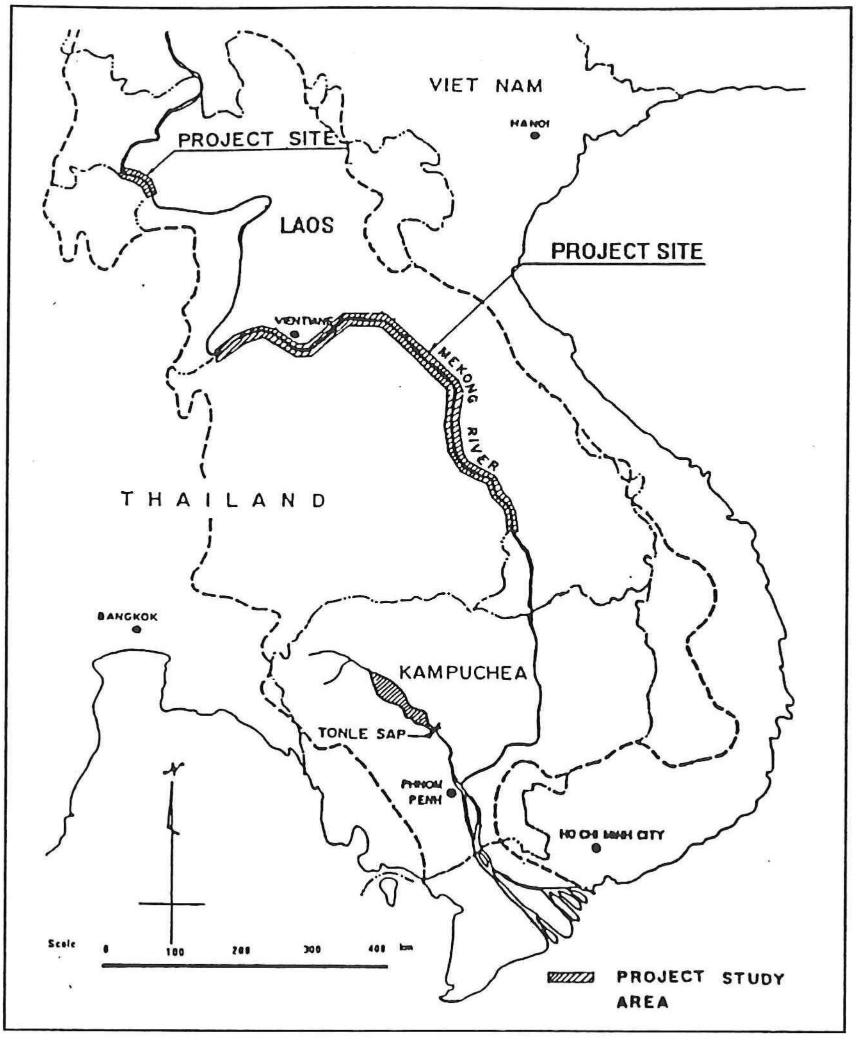
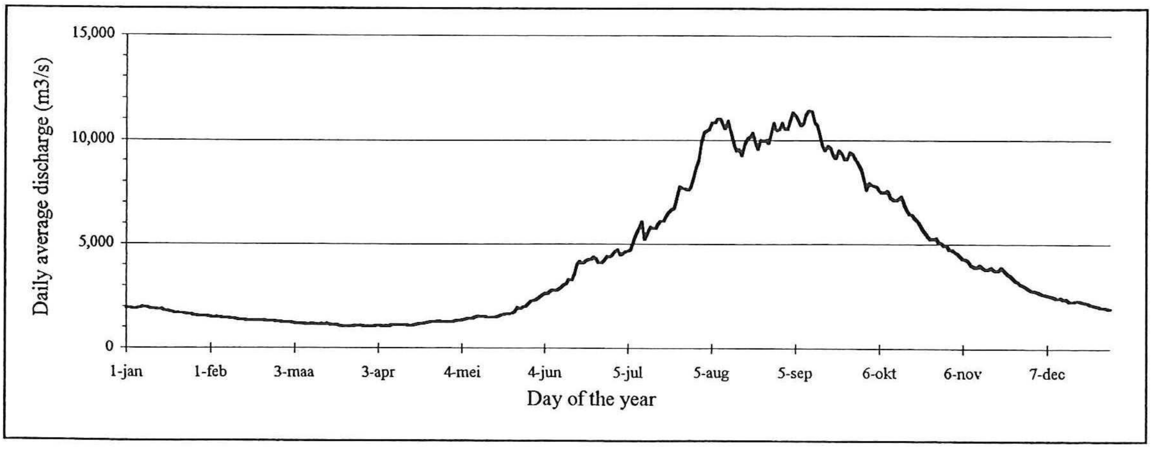


Figure 2 Mekong River and location of the project area (river bank on the Thai side)



LEGEND: ---- .. ----- country borders
- - - - - - - - catchment area

Figure 3 Mean annual hydrograph (1971-1990), Mekong river at Nong Khai (km 1550)



14. Mekong River Bank Erosion Study

The Mekong river is the largest river in Thailand and one of the largest rivers in the world. It originates some 5,000 m up in the Himalayas on the Tibetan Plateau in China and flows through Myanmar, Laos, Thailand, Kampuchea and Vietnam where the river discharges into the South China Sea (annex B, figure 1). The total basin area is about 795,000 km^2 (annex B, figure 2) and the total length of the main river is about 4,200 km. The river conveys, on average, some 200,000 million m^3 of water and 150 million tons of sediment annually.

The runoff in the Mekong is composed of snowmelt and rainfall. The upper basin which constitutes 23% of the total basin area, contributes 20% of the annual runoff, largely from snowmelt, while the rest of the runoff is from rainfall. Therefore, the pattern of the flow in the Lower Mekong River Basin reflects to a large extent the pattern of rainfall during the year, see Figure 3. The climate is tropical and dominated by two monsoons. The Southwest monsoon, or the rainy season, normally affecting the area from mid-May to early October, is predominant when atmospheric pressures are comparatively low over Southeast Asia. The Northeast monsoon lasts from early November to mid-March.

The river is the international border between Thailand and Laos over a length of 920 km, split into two reaches which are also the project areas of the Mekong River Bank Erosion Study, see Figure 2. The first reach is 85 km long in the 'Golden Triangle' area in the Chiang Rai province in northern Thailand, the second reach is 835 km long from the Loei to Ubon Ratchathani province in north-eastern Thailand.

Along the project area, the Mekong river has a variable width and is controlled by the geology at many points. Almost along the whole length of the river the local geology and rock outcrops are very important. Floodplains are only marginally developed along most of the Mekong river reaches and the river reaches the flood plain levels only during extreme floods. Furthermore, only the reach near Nong Khai has been identified as fairly alluvial (a reach where the influence of the bed rock outcrops appears to be almost negligible).

Like many rivers, the Mekong river erodes its banks at many locations. This erosion has not reached alarming proportions but considerations on: (i) international border, (ii) sustained socio-economic development of the region, (iii) protection of existing infrastructure and cultural heritage requires prompt attention at critical locations along the waterfront. There are seven provinces along the Mekong river in Thailand and the urban housing and infrastructure will become much more important than in the past, adding significantly to the value of the property and services needing protection from erosion of the river banks. Current projections suggest that at critical locations the average rate of erosion of the river bank is 2-5 meters per year.

Taking this into account, the Public Works Department (PWD) of the Royal Thai Government initiated the Mekong River Bank Erosion Study (MRBES). The study aims at developing an integrated strategic plan to counteract the problems induced by river bank erosion. The consultants group consisting of NEDECO/HASKONING, Span and WDC, commenced the project assigned to them by collecting and processing the available hydraulic, hydrologic, geologic, geomorphologic and environmental data.

Special survey programs were initiated to observe bank erosion during the high flow season of 1994 and to obtain data necessary for the detailed design of river bank protection works. The surveys included: bathymetric surveys, velocity measurements, bed sampling and geotechnical investigations of the river banks. The compiled dataset will allow the determination of the river characteristics necessary to draft the strategic plan and to design river bank protection works for selected sites. The project commenced in March 1994 and will be finalized in the beginning of 1995.

15. Uncertainty of 'observed parameters'

One of the elements of the probabilistic design approach is the analysis of available measurements of certain parameters. These measured parameters will be called 'observed parameters'. Of some of these parameters large time-series are available, for example the waterlevels, discharges and wind velocities in the hydrological analysis in the MRBES. These data-series can be used to derive probability distributions of normal and extreme events, provided the series are stationary, homogeneous and relatively consistent, which can be checked with several statistical tests.

For other parameters like the suspended sediment transport, also time-series are available. Because this data depends on more uncertain factors, the (absolute) consistency should be checked before using the data in a probabilistic analysis like with the waterlevels. For a spatial series of a parameter it is necessary to check the homogeneity of the stretch of river. To illustrate this, the measured river width is taken as an example.

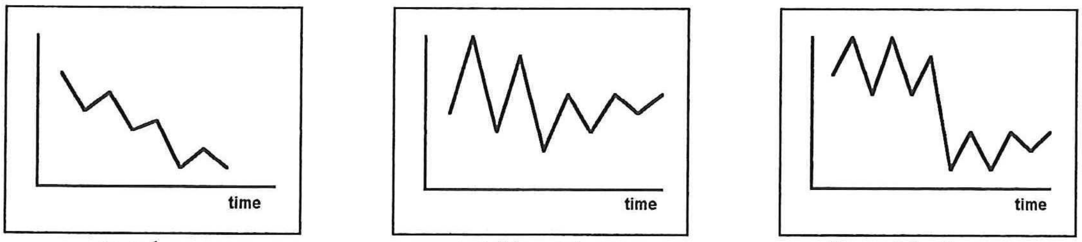
If there is hardly any information available about a certain parameter used in the calculations, and even the information available is surrounded with uncertainty it is still possible to make an assessment of the uncertainty with the use of 'simple' probability distributions.

15.1. Statistical validation observed data

Daily average waterlevel and discharge data, collected both on paper and on diskette, were stored with the associated gauge information (datum, location etc.) in the database HASDAT (proprietary software HASKONING), which was then used to generate annual maximum values, monthly averages etc. The available waterlevel data of 16 stations (annex B, figure 3) along the Mekong river included a fifteen year common period from 1976 through 1990. The available discharge data of 8 stations along the Mekong river included a twenty year common period from 1971 through 1990.

If hydrological data is used in frequency analysis or system simulation it should be stationary, consistent and homogeneous. To check these conditions, the data can be screened by using statistical tests on annual or seasonal time-series. This includes testing the absence of trend, the stability of mean and the stability of variance. See Figure 4 for these types of non-stationary series.

Figure 4 Three non-stationary time-series

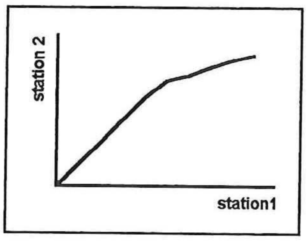


trend | unstable variance | 'jump' in the mean

Trend analyses were performed using the Spearman trend analysis. The stability of mean and variance were tested using respectively the standard t- and F-test. All these tests were carried out on the mean annual discharge. The Spearman trend analysis doesn't require the assumption of an underlying statistical distribution. When using the t- and F-tests, the samples are assumed to be normally distributed. But even for samples that are not from a normal distribution, the F- test gives a reasonable indication of the stability of variance and the t-test a good indication of the stability of mean, provided the sub-series have equal length.

The commonly used method to check relative consistency of data records of neighboring stations, the double mass analysis, was applied on the discharge data. See Figure 5 for an example of the double mass curve. If there is a significant break (change of slope) in the double-mass line, the stations are not consistent.

Figure 5 Double Mass curve



In Annex B, BOX 1, all these methods of data checking are briefly explained.

15.2. Frequency and duration curves

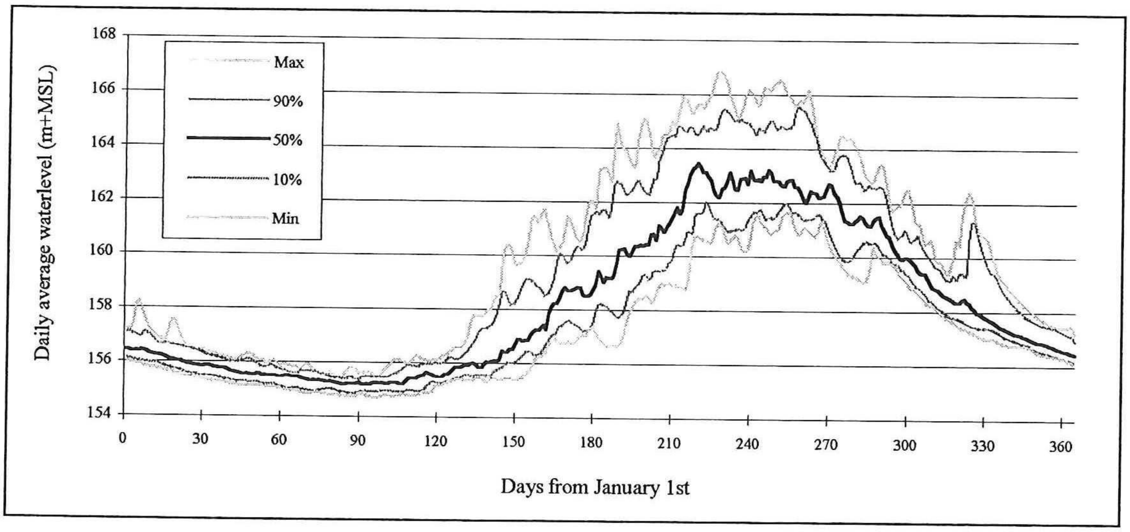
The variation of waterlevels and discharges throughout the year can be represented by frequency curves, where each curve indicates the waterlevel or discharge for a specific probability of non-exceedence. Related to this are duration curves. These are the average number of days that a certain waterlevel or discharge is not exceeded. These curves are useful for the computation of bed scour in the morphological study, planning construction window and determining navigability.

Frequency curves are established in the following way (Jansen, 1979): Given a record of several years of data, for each day in the year a cumulative frequency distribution is made:

F(Waterlevel | day of the year)

By connecting the values of equal non-exceedence probability for the successive days of the year, often at a 10, 50 and 90 percent probability level, the frequency curves are obtained. Figure 6 shows the frequency curves of the waterlevels of Nong Khai (km 1550) as an example.

Figure 6 Frequency curves waterlevels at Nong Khai (km 1550), 1971-1990

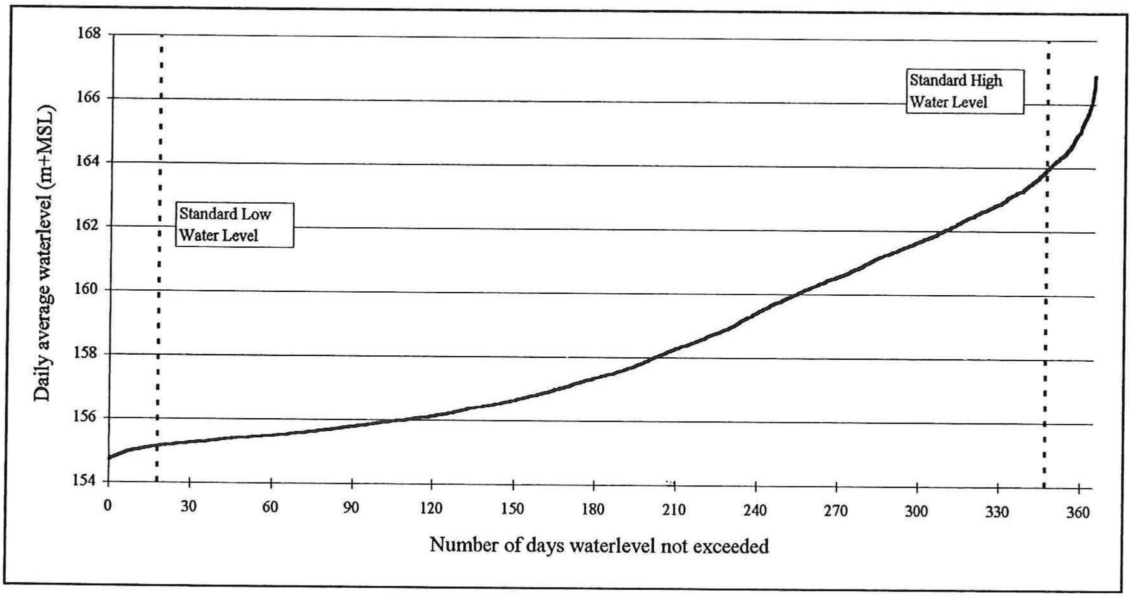


Duration[1] curves are ranked representations of the frequency curves (Jansen, 1979). The duration curve gives the average number of days that a given value was not exceeded in the years considered:

F(Waterlevel) * 365 days

Figure 7 shows the duration curve for Chiang Saen (km 2364) as an example. These curves are for example used to derive the Standard High Water Level (SHWL = the waterlevel not exceeded during 18 days per year (5% of the year)) and the Standard Low Water Level (SLWL = the waterlevel exceeded 347 days per year (95% of the year)). These are useful waterlevels in assisting a contractor to define a construction window for planning his activities and are often shown on construction drawings. The definition for these waterlevels are:

Figure 7 Duration curve waterlevels at Nong Khai (km 1550), 1971-1990



[1] the name 'duration curve' is rather misleading because the number of days that a certain waterlevel or discharge is not exceeded is a sum of individual observations that do not necessarily form a closed period

BOX 1 Probability distributions extreme values

Some of the most frequently used probability functions for maximum extreme values.

Gumbel (Extreme Value distribution, type I (EV1))

$$f(x) = \frac{1}{\alpha}\exp\left(-\frac{x-u}{\alpha} - \exp\left(-\frac{x-u}{\alpha}\right)\right) \qquad F(x) = \exp\left(-\exp\left(-\frac{x-u}{\alpha}\right)\right)$$

Moments: $\mu = u + 0.577\alpha$ $\quad \sigma = \alpha\pi/\sqrt{6}$ $\quad C_s \approx 1.14$ $\quad C_k \approx 5.40$

Weibull (Extreme Value distribution type III (EV3), originally developed for minimum values)

$$f(x) = \frac{\beta}{\alpha}\left(\frac{x}{\alpha}\right)^{\beta-1}\exp\left(-\left(\frac{x}{\alpha}\right)^{\beta}\right) \qquad F(x) = 1 - \exp\left(-\left(\frac{x}{\alpha}\right)^{\beta}\right)$$

Moments: $\mu = \alpha\,\Gamma(1+1/\beta)$ $\quad \sigma^2 + \mu^2 = \alpha^2\,\Gamma(1+2/\beta)$

(The Extreme Value distribution, Type II (EV2) or 'Fréchet' distribution can be obtained by substituting $-\beta$ for β)

Three-parameter Lognormal (LN 3)

$$f(x) = \frac{1}{x\,\sigma_y\sqrt{2\pi}}\exp\left(-\frac{1}{2}\left(\frac{\ln(x-u)-\mu_y}{\sigma_y}\right)^2\right) \qquad \text{with } y = \ln(x-u)$$

Moments: μ_y and σ_y

(The two-parameter Lognormal (LN2) can be obtained if u is set to zero)

Log Pearson (LP3)

$$f(x) = \frac{\lambda^{\beta}(y-u)^{\beta-1}\exp(-\lambda(y-u))}{x\Gamma(\beta)} \qquad \text{with } y = \ln(x)$$

$$\Gamma(\beta) = \int_0^{\infty} x^{\beta-1}\exp(-x)dx \qquad \text{(Gamma function[2])}$$

Moments[3] : $\mu_y = u + \beta/\lambda$ $\quad \sigma_y = \sqrt{\beta}/\lambda$ $\quad C_{s,y} = 2/\sqrt{\beta}$

- The CDFs, F(x) are written as the **non-exceedence curve** (F(x) is the probability that x is not exceeded). To arrive at the **exceedence curve** one can simply take: 1 - F(x). The PDF's form does not change as a result of this transformation, it only becomes negative: f (x) → - f (x).
- Minimal extreme value distributions can be obtained for all the distribution types by substituting x with -x.[4]
- For all formulas: x = variate, y = ln (x)
  α = factor, λ = 1/α
  β = exponent
  u = deviation from x
  μ = average
  σ = standard deviation

[2] In Vrijling (1987), b3 (lecture notes), this function is written incorrect

[3] In Ven te Chow (1988) p.373, the moments for the Log Pearson Type III distribution are incorrect. Instead of the parameter λ, 1/λ should be used.

[4] This transformation is easily mixed up with the transformation of non-exceedence to exceedence curve, mentioned earlier.

15.3. Frequency analysis extreme events

A frequency analysis was conducted to determine the return period (T) for extreme values of the water levels[5] for eight main stations along the Mekong river. The annual maximum gauge levels were retrieved from the database HASDAT containing daily average gauge levels.

The frequency analysis consisted of the following steps:

1. The first step is the choice of the possible probability distributions and the choice of the method of fitting. In BOX 1 some of the most frequently used probability functions for extreme values are shown. This first step is to find the parameters for which each selected distribution function fits the data best. To do this fitting, four methods are available:

- Method of moments
- Linear Regression (least-square method)
- Maximum Likelihood
- Bayesian Parameter Estimation

PLOTTING POSITION

An important aspect of the linear regression fitting procedure is the choice of the method of determining the plotting position of the measured extreme values: what is the return period of the observations? For every distribution a different method of plotting position is the most suitable, representing the expected value for every observed value for that distribution. Because for many distributions this expected value can not be solved analytically, practical methods have been developed. Because the fitting was done with the Maximum Likelihood method, this was not an issue here. But if the various fitted distributions are plotted in one graph and the best fit is determined with the eye, the choice of plotting method also has some influence. In Figure 8 the Cunnane method of plotting was used, defined as:

$$y_i = \frac{i - 0.4}{N + 0.2}$$

where y_i is the ordinate of the ordered statistic (the probability of exceedence). To put this value in a graph it is transformed to the reduced normal variate, k(N). This can be calculated with the inverted normal distribution with ($1-y_i$) as input (the probability of non-exceedence).

The fitting was done with the computer program 'Consolidated Frequency Analysis (CFA)' which provides the 'method of moments' and the 'maximum likelihood' method of fitting. The Maximum Likelihood Method was chosen to fit the following probability functions to the sample data:

- Gumbel or Extreme Value Type-I (EV1)
- Log-Normal 2-parameter (LN2)
- Log-Normal 3-parameter (LN3)
- Log-Pearson Type-III (LP3)

[5] Using waterlevels for frequency analysis for extreme events should be done with care because in case of bank overflow the form of the curve can significantly change and will most probably be upper-bounded. For the Mekong river, however, this is not the case for the return periods considered.

The results obtained are illustrated in Figure 8. In Table 1 the observations are shown and in Table 2 the distribution parameters obtained for the best fit of each distribution.

Figure 8 Frequency analysis at Nong Khai (km 1550), 1976-1990

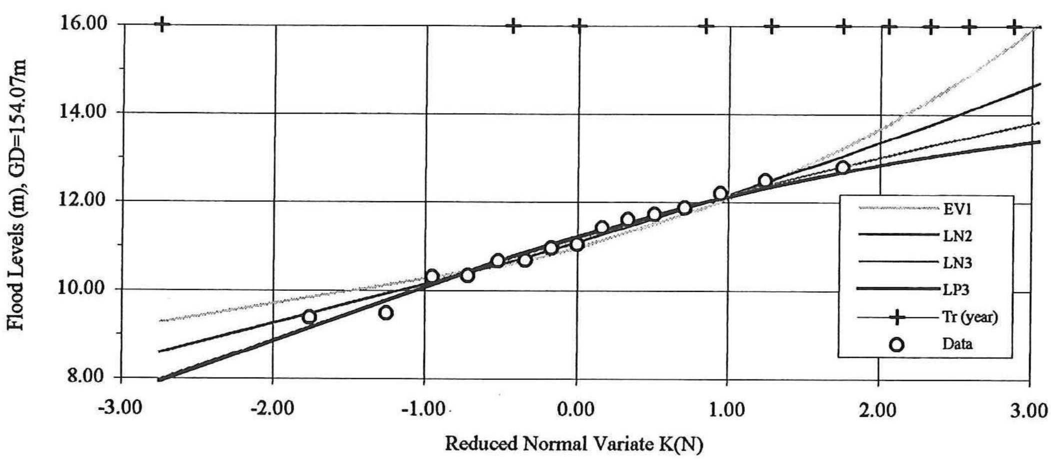


Table 1 Maximum waterlevels Nong Khai (km 1550; GD 153.7 m), 1976-1990

Year	'76	'77	'78	'79	'80	'81	'82	'83	'84	'85	'86	'87	'88	'89	'90
Max. level	11.7	10.3	12.8	11.0	12.5	11.6	11.4	10.7	11.0	12.2	10.7	9.4	10.3	9.5	11.9

Table 2 Distribution parameters

EV1		LN2		LN3		LP3	
α	0.796	μ_y	2.406	μ_y	2.307	λ	6.538
β	–	σ_y	0.093	σ_y	0.101	β	-0.036
u	10.668	u	-	u	21.225	u	14.050

REQUIRED ACCURACY

During the analysis, fitting the distributions on the gauge levels proved to yield significantly different results than a fit on the absolute waterlevels expressed in meters above Mean Sea Level. This should theoretically make no difference, since it simply means shifting the reference level. Whether this was caused by problems with required accuracy of the program CFA was not identified, but it is the most likely cause. The frequency distributions have finally been fitted to the gauge height data instead of the absolute waterlevels for maximum accuracy.

II. The second step is the choice of the model. The main question is: Which distribution in Figure 8 fits the data points best? Several tests are available to test the goodness of fit of a model to datapoints, such as:

- Chi-square
- Kolmogorov-Smirnov
- Anderson-Darling test

In annex B, BOX 2, these tests are briefly explained. These tests determine whether the fitted distribution should be accepted or rejected, based on a specified confidence level, for example 90%. Because all of these three tests have rather wide confidence limits, all distributions in Figure 8 will be accepted. Therefore it was preferred to choose the model by visual goodness of fit.

Figure 8 shows that all frequency distributions fit the data records reasonably well. In general we can see that the three-parameter distributions, LN3 and LP3, fit the data better than the two-parameter distributions, as was expected since the three parameter functions are more flexible and can take on more different shapes. The Log-Pearson type III distribution (see BOX 1) which is frequently used to calculate frequency distributions of hydrologic data, was finally adopted as the function type to be used.

THE BEST FIT ?

The main problem of choosing the most suitable distribution is that most of these distributions will fit reasonably well through the bulk of the points with low return periods but that the part we are mainly interested in for prediction of future events is the tail. Here the values predicted by the different distributions can vary much more, in the example the difference between the EV1 and the LP3 distribution is about 2m for a return period of 100 years!

Another aspect is that this fit is based on only 15 data points (the extreme waterlevels of each year) which is a relatively short data-series. The statistical uncertainty can be significant. From other research it shows that if data-series of the same length are generated, assuming the distribution which gave the best fit, the variation of events in the tail of the distributions can be considerable. This is the *statistical uncertainty* mentioned in section I.

If an engineer would like to base his choice on the results of the 'goodness-of-fit' tests, which of the tests described above would be the most objective indicator and what is a significant difference between indicator values for different distributions? These tests usually have very wide confidence limits, and therefore many of the distributions would not be rejected by these tests and are therefore equally suitable.

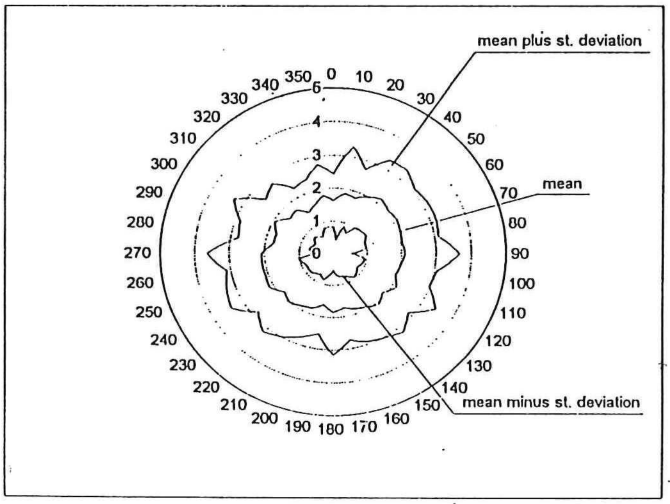
An alternative way to define the extreme events would be to determine what the absolute difference between "equally" good fitting distributions is for a certain return period. From this, a range could be derived of the most probable values for this extreme event instead of one single design value for every return period. The probability distribution over this range could be taken as uniform or a truncated normal distribution.

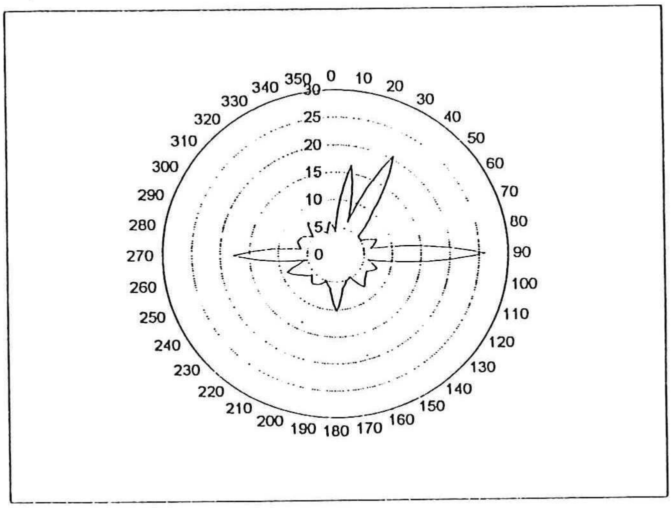
15.4. Wind data

The collected wind data consists of 3-hourly average wind velocities and -directions, observed at the Nong Khai (km 1550) meteorological station for the period November 1982 through December 1993. The dominant wind direction appeared to be easterly (= 90 degrees in the graphs) with an average wind velocity of 7.4 km/hr or 2 m/s. The maximum wind velocity observed was 94.5 km/hr or 26 m/s. The mean and maximum wind velocities and the corresponding directions are shown in Figure 9.

To determine the design wind velocity for 50 and 100 years return period, a Gumbel distribution was fitted to annual maximum 3-hourly average wind velocities. The 100-year wind is easterly and has an corresponding velocity of 26.3 m/s

Figure 9 Mean wind velocities and maximum wind velocities





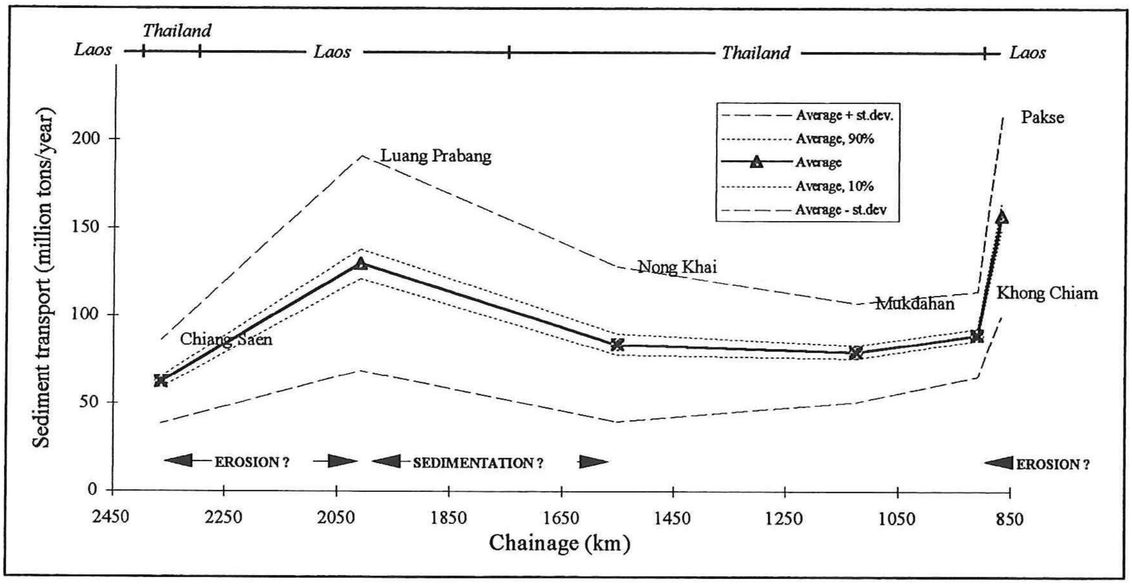
15.5. Consistency sediment transport

The sediment transport in a river is determined by the hydraulic conditions and by the characteristics of the bed material. From the present knowledge of the characteristics of the bed material in the Mekong (fairly fine sand with D_{50} = 300 μm), it can be concluded that the sediment is always in motion and largely in suspension (Shields parameter >> 0.03 and suspension parameter > 1).

Because the measurements of this sediment transport involves the uncertainty of the measurements of the discharge and the measurements of the suspended sediment concentration, it is necessary to check the absolute consistency between the stations before performing a probabilistic analysis as was done with the waterlevels and discharges. Figure 10 shows the average annual suspended sediment load between 1970 and 1989[6]. In this figure also the two-sided 90% confidence limits on the average are given and the standard deviation from the mean to both sides is indicated.

For a 'stable' river stretch the yearly sediment transport can be expected to increase with increasing discharge in downstream direction. According to Figure 10, however, there were river stretches with considerable erosion and sedimentation in this 20 year period. This is not confirmed by the geometrical study of cross-sections during this period. The reason for this discrepancy might be found in the fact that the information of the two stations in Laos (triangular marker) comes from a Laotian organization and the information for the other stations in Thailand (square marker) comes from a Thai organization. The methods of measuring discharge and suspended sediment concentration might differ between these two organizations. The two stations in Laos report considerably higher sediment transports with a greater variance over the years than the four stations in Thailand. Independently, the two stations in Laos and the four stations in Thailand confirm the increase of sediment transport in downstream direction and could be used for an analysis as for the waterlevels in the previous paragraphs.

Figure 10 Average annual measured suspended sediment load (1970 - 1989)



[6] data from Harden & Sunborg (1991)

15.6. Homogeneity spatial-series river width

Spatial data-series can also be used in probabilistic analysis, providing the underlying mechanism is stochastic. For certain morphological parameters this might be the case for example the riverwidth. Although no theoretical foundation has been found, the equation

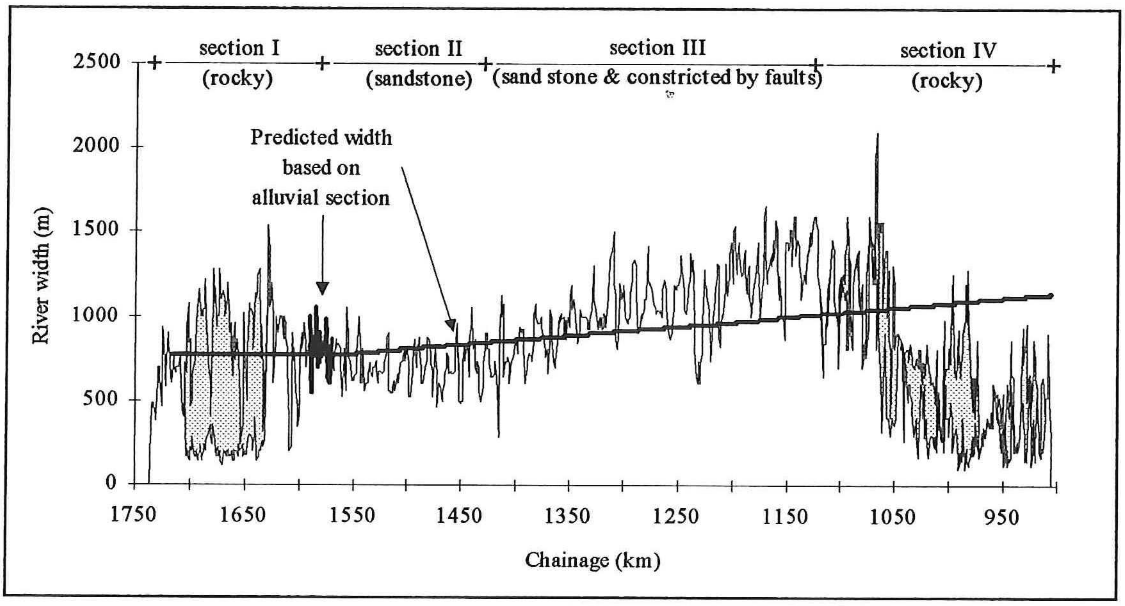
$$B = a\sqrt{Q_{bf}}$$

appears to be valid for a variety of alluvial environments. B is the riverwidth (m), Q_{bf} is the bankfull discharge (here assumed to be $Q_{1.5}$) and a is a proportionality factor, which varies for different rivers depending on bank material characteristics and the presence of vegetation. If we assume that this relationship determines the width of the river, but that on top of that several other unidentified mechanisms cause a stochastic variation in time around this value, this would become

$$B = NORMAL(a\sqrt{Q_{bf}}, \sigma)$$

In Figure 11 the width of the Mekong River, measured from the bathymetric maps, is given as a function of the chainage for the reach Chiang Khan-Khong Chiam. In two rocky sections two lines are drawn. The upper line represents the total width of the river and the hatched area in-between is protruding bedrock outcrops, that are exposed during low flow conditions but are drowned during higher stages.

Figure 11 River width, longitudinal overview (km 1750 - km 900)



In the analysis it was found that for the Mekong River only a stretch of 30 km near Nong Khai (km 1550) is really alluvial. By using the average discharge and the average width of this stretch, the proportionality factor could be derived which was found to be 6.4 . The theoretical width for the complete stretch, if it had been alluvial, as well as the alluvial river stretch are indicated with a bold line in Figure 11. The morphological analysis in the MRBES indicated that the influence of local and global geology was the most important factor for the planform characteristics.

DETERMINISTIC & PROBABILISTIC METHOD

If a river is alluvial, predicting planform characteristics can be done based on the deterministic morphological models. An alternative approach is a probabilistic analysis of aerial photographs and satellite images. In that case, parameters such as the sinuosity, the radius of the meanders and the meanderlength can all be measured for homogeneous river reaches (with the same slope and discharge-range) and sorted in classes. The resulting histograms of these classes can be interpreted as the approximated PDF's of the considered parameters.

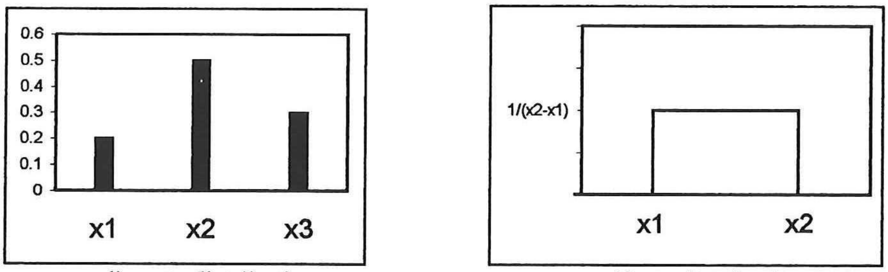
15.7. Uncertainty in case of very limited data

One of the critical remarks on using probabilistic design methods that the author encountered during this thesis study, considered the situation that there is hardly any information available on a certain parameter. In these cases the probability distribution of this parameter can not be derived as for the examples presented in the previous paragraphs, and therefore a probabilistic approach would not create any added value in these cases, according to these critics.

This is a rather remarkable criticism because a probabilistic approach has specifically been developed to account for uncertainty and should therefore be especially useful in these situations of limited data. The problem is, however, that a designer might have the impression that he/she is forced to describe the parameters with probability distributions functions that are too 'sophisticated' for the information available. If three estimates of a parameter are available it doesn't seem realistic to base normal distribution or an extreme value distribution on such few points.

But it is also unnecessary. If it is estimated, for example, that a parameter's state of nature is somewhere in a range between x_1 and x_2, and there is no reason to assume that any value in this range is more probable than any other, a normal distribution is not the most suitable model. In such a case, a uniform distribution is a better model. Or, like is frequently used in economic forecasts, an engineer could express (future) expectations in a 'worst-case' / 'best-case' approach, with discrete values and estimated probabilities for each value.

Figure 12 Discrete and uniform probability distribution



discrete distribution

uniform distribution

One of the reasons that these distributions are not often used lies in their applicability in the probabilistic calculations. For level II probabilistic calculation all distributions are approximated with a normal distribution. For the discrete distribution this is hardly possible. For the uniform distribution, this approximation is possible, but because of the different form of the distribution it could lead to a calculated probability of failure that is significantly different from the real probability of failure

16. Uncertainty of derived variables

If a parameter is a derived parameter and can be calculated through a functional relationship with one or more 'observed parameters' with known probability distributions, the approximate probability distribution can be derived through probabilistic calculation.

An example was already shown in section II where the probability function of the current velocity was determined with the equation $u = C\sqrt{hi}$ and a Z-function of the form $Z = u - C\sqrt{hi}$.
By varying the values for u and calculating the probability of exceedence for each value, a Cumulative Distribution Function (CDF) could be obtained. The difficulty in this method, besides determining the distribution function of the independent parameters, is to estimate the uncertainty of the velocity, u, which should reflect the model uncertainty (see section I) of this functional relationship.

If the parameter is not a direct function but determined by calibration in an iterative or a graphical way, this direct calculation method is not applicable. In these situations, an estimation for the uncertainty of the derived variable can be determined through a trial-and-error method, or **sensitivity analysis**. By redoing a calculation for different values of the unknown parameter and (subjectively) deciding which values result in a satisfying calibration, a range is obtained of possible values of the parameter. This is the main difference between direct probabilistic calculation where the uncertainty and probability of failure is the 'objective' output of a calculation. In case of deriving uncertainty with a sensitivity analysis, the engineer has to make a subjective choice.

One of the most important aspect of such an analysis is the flexibility to vary and change data and recalculate the results in the analysis tools. Based on these results the engineer will have to make decisions about the uncertainty of derived parameters. The goal is to provide the engineer with 'analysis tools' with which to make the most time-efficient analysis with as much flexibility as possible.

16.1. Hydraulic gradients

16.1.1. Graphical check gauge datums

All the gauges along the project area are staff gauges with a variable number of staggered staff sections. Gauge datums were obtained from the Hydrologic Yearbooks (NEA, 1990). The gauge datums were roughly checked by interconnecting gauge readings of all stations for a specific day with low discharge, and evaluating whether the obtained hydraulic gradients were reasonably in accordance with a hydraulic profile available for lowest low water (LLW)[7]. After this check there was reason to believe that some of the gauge datums were not correct. It was decided to check a selected number of gauge datums by means of a topographical survey[8]. During this survey the gauge datums of 7 stations had to be adjusted, some of them by 0.25 to 0.40 meter, but others by 4 to 10 meters!

[7] This had been established together with a bathymetric map for navigation purposes
[8] In a topographical study, the gauge datums are checked with benchmarks: concrete "fixed" points with known elevation above the reference level (most often Mean Sea Level (MSL))

CORRECTING DATA

What is the implication of this for the reliability of the data of these stations? Can the data from a station of which the gauge datum has been corrected, be trusted? The large differences of several meters can only be the result of a measurement or reporting error. The small differences, however, could also have been caused by a sudden or gradual change in position of one or more of the staggered staff gauges. Only at some of the stations these staff gauges were placed on a concrete foundation. At all other stations the staff gauges were directly placed in the river bank. At some stations staff gauges were even used as poles to tie fishingboats onto.

Although we might not be able to pinpoint a certain cause for the discrepancies encountered, we should communicate the fact that there is an uncertainty surrounding the data from these stations, even for the ones that have been corrected. If quantifiable, an **error bar** (relative to the data point-value or an absolute error) could be added to the data point of that station. But if not quantifiable, even a **color code** for these data would in this case help to communicate this uncertainty and help to signal possible errors more quickly in further analysis of this data. An example:

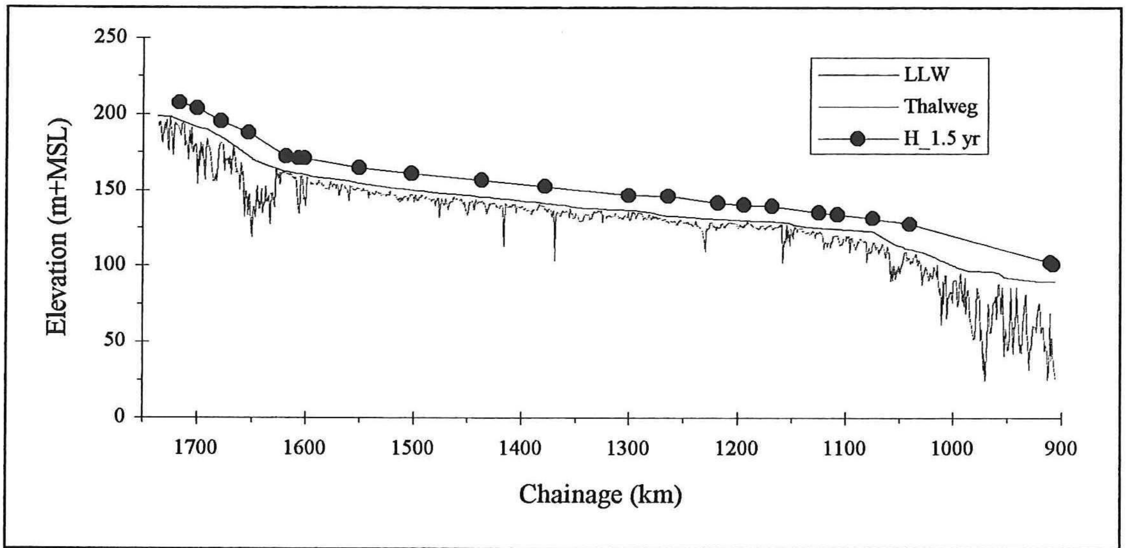
- Red: The gauge datum was not checked
- Orange: The gauge datum was checked and corrected,
- Green: The gauge datum was checked and proved to be right

Both methods, error bars and color codes, are easily applicable in, for example, spreadsheet-programs.

16.1.2. Hydraulic profile

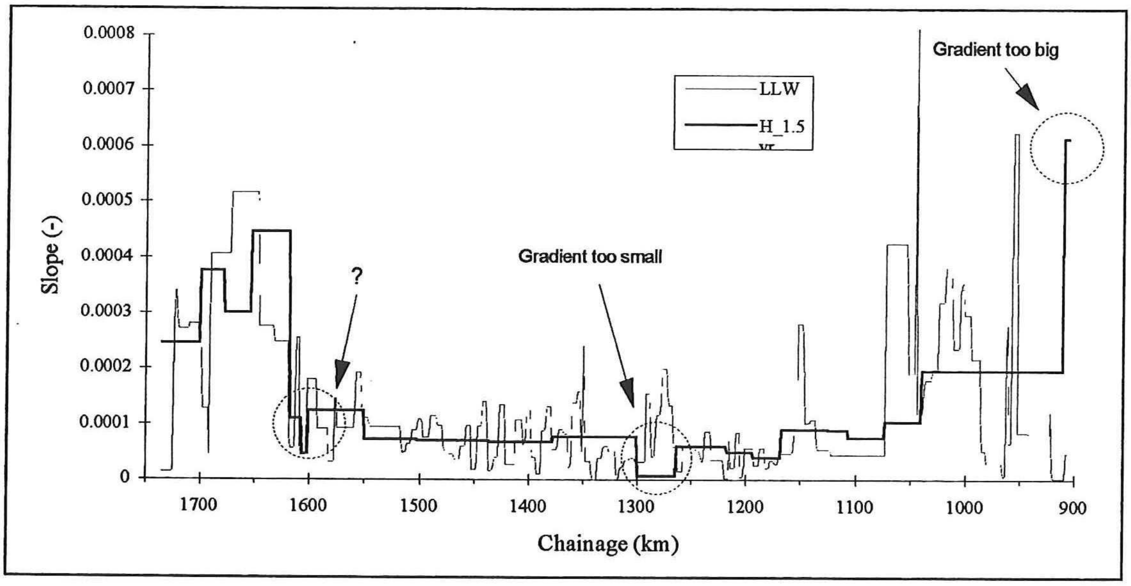
The waterlevels for the specific return periods at the various stations can be interconnected to obtain a longitudinal profile of flood levels along the river see Figure 13. The Lowest Low Water levels were available for every 1 km. from a map made for navigational purposes. The profile for a return period of 1.5 years (important for the geo-morphological analysis as it was assumed to be the bankfull discharge) was determined for 22 stations along the river. These are marked with black dots in Figure 13.

Figure 13 Thalweg level and waterlevels, longitudinal overview (km 1750 - km 900)



The most obvious check for the consistency is that the water should flow in downstream direction, which it does. For a more detailed analysis, we should look at the graph for the hydraulic gradients. In this study the hydraulic gradient is defined as the difference in waterlevel between two consecutive stations divided by the difference in chainage. In Figure 14, the hydraulic gradient is shown for LLW and for the waterlevel with a return period of 1.5 years.

Figure 14 Hydraulic gradient, longitudinal overview (km 1750 - km 900)



The gradients for LLW show a great variation whereas the gradients for $H_{1.5}$ vary much less. Part of this difference is explained by the fact that for $H_{1.5}$ less points are available and the in-between values are interpolated. Local maximum and minimum values remain therefore undetected.

But this difference is also one of the indications of the influence of bed outcrops in the Mekong river. The steep slopes in some reaches at LLW, are caused by local energy losses due to the bedrock. This could be the transition from supercritical flow to subcritical flow, the retardation of the flow velocity after constrictions (Carnot losses) or bedrock outcrops that act as roughness element, and therefore are responsible for additional energy losses on top of the alluvial roughness (particle- and bedform roughness). At high discharges the influence of these bedrock outcrops becomes less pronounced and many of the small 'rapids' are 'drowned'.

From Figure 14 it shows that the hydraulic gradient between Pak Hai Lang Ka (km1300) and Ban Chai Buri (km1264) is unrealistically small, especially compared to the hydraulic gradients at low flow in this stretch (see Figure 14). This is most probably due to an erroneous gauge datum at Pak Hai Lang Ka (which was not checked) or at Ban Chai Buri, of which the gauge datum was adjusted with 9.31m. At Khong Chiam (km 920) the gradient is too big. For LLW the gradient is almost negligible at that station. For some other stations, the gradients were also doubtful but not pronounced enough to correct them.

To determine the uncertainty of the hydraulic gradient in a certain reach, the uncertainty of the gauge datum level, Z, and the extreme waterlevel is needed of the two stations bounding that reach:

$$H_1 = \text{UNIFORM}(Z_{1,min}, Z_{1,max}) + \text{LP3}(\lambda_1, \beta_1, u_1)$$
$$H_2 = \text{UNIFORM}(Z_{2,min}, Z_{2,max}) + \text{LP3}(\lambda_2, \beta_2, u_2)$$
$$i_{1\text{-}2} = H_1 - H_2$$

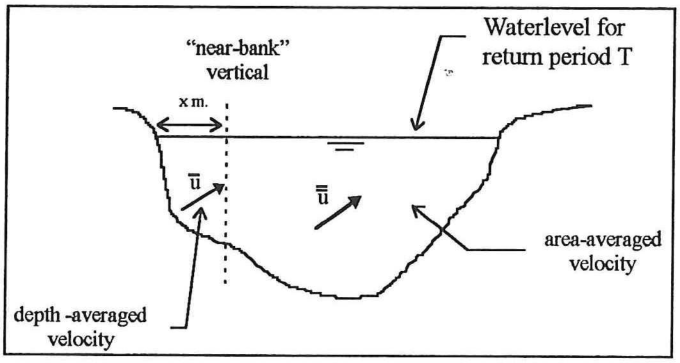
It will be too much work to do this for every interval between stations, but even analyzing one or two intervals will give an idea of the order of magnitude of this uncertainty.

16.2. Velocities

One of the most important boundary condition for the design of the bank protection is the maximum flow velocity near the bank. The near-bank velocity is not easy to derive, as it depends on a number of factors, notably the planform of the river and the shape of the cross-section. The following approach was chosen to derive these velocities:

1. With the available **cross-sectional data** and **rating curves** at the eight main hydrological stations, the **area-averaged velocities** were derived for selected return periods.
2. About 400 cross-sections were measured from the bathymetric map. By interpolating the discharge between the main stations, it is possible to derive the area-averaged velocities at these cross-sections as well, by proceeding as in step 1.
3. At Nong Khai, **velocity measurements** were available. Here it is possible to obtain area-averaged velocities and depth-averaged velocities directly from the measurements, and to derive a ratio depth-averaged/area-averaged velocity over the cross-section and over the year.
4. As 3., for additional velocity measurements which will be carried out during the 1994 flood season.
5. With the information obtained in 3. and 4. it is possible to derive near-bank design velocities at the proposed protection sites by interpolation of the ratios and applying them to the area-averaged design velocities obtained in 2., taking (geo-)morphological and geotechnical aspects into account.

Figure 15 Definition of the used velocities



16.2.1. Reported rating curves

In the hydrological yearbooks rating curves were provided for eight main stations, by the National Energy Administration (NEA). Up till 1975, rating curves were made every year, but after that, the rating curve from 1975 was used. This was most likely the result of the ending of the Indochina war in 1975, when Cambodia, Laos and Vietnam dropped out of the Mekong Committee and the plans for the construction of dams in the Mekong were indefinitely postponed[9]. With this, the need to have a sound monitoring of discharges seemed to have disappeared.

Regression analyses were performed to fit the 1975 rating curves using a fourth order polynomial:

$$Q(h) = a_0 + a_1 h + a_2 h^2 + a_3 h^3 + a_4 h^4$$

with Q = discharge [m3/s]
a_i = coefficient of regression analysis [-]
H = gauge height or stage [m]

The analysis found a very good fit for such a polynomial for every station. Values for the r-squared (r^2), which is a measure for this goodness of fit[10], range between 0.98 and 0.99. This confirms the expectation that the reported rating curves are already lines that are fitted to the measured samples with similar fitting techniques. The question is whether these rating curves are credible, based on the cross section. From a theoretical point of view they are.

For uniform flow the stage-discharge relationship will be of the form:

$$Q = A\bar{u} = \frac{1}{n} A R^{2/3} \sqrt{i}$$

with Q = discharge [m^3s^{-1}]
A = wetted area [m^2]
R = hydraulic radius [m] = A/P
P = wetted perimeter [m]
u = area-averaged velocity [ms^{-1}]
n = Manning resistance coefficient [$sm^{-1/3}$]
i = hydraulic gradient [-]

The parameters A and R are both functions of the stage. If the stage increases, A and R will increase. The parameters i and n will probably also vary with the stage: n tends to decrease with increasing stage, the effect on i is less clear, and especially for the Mekong river it depends largely on the local geology at each location. In this study, the parameters n and i are taken as constant values for all stages.
Providing the shape of the cross-section is not too unregular and floodplains are hardly present, which is the case along the Mekong river, the parameters A and R, and therefore the discharge Q, will be a rather smooth and strictly crescending function of the stage. Such a function can usually be approximated very accurately with a polynomial.

Although most of the stations report that flow measurements were made at all stages, for some stations it is more likely that flow measurements were done at low and medium flows whereafter the

---

[9] The Mekong Currency, Liesbeth Sluiter, Bangkok, 1992.

[10] The correlationcoefficient is defined as $r = \frac{cov(x,y)}{\sqrt{var(x)\,var(y)}}$

rating curve was constructed through extrapolation with the polynomial. This became clear after checking the cross-sections of some of the places were rating curves were established. Some rating curves were extrapolated to levels that were never reached during the years or to levels above the banks of the cross-section, without any change in the form of the rating curve.

16.2.2. Measured cross-sections

From a bathymetric map (see example Figure 16), the cross-sections could be derived. The depth soundings, indicated in the map, were put in a spreadsheet and a graph was made by interconnecting these readings. The last depth readings were connected with the level of the thick black lines in the bathymetric map that indicate the river bank. Almost along the entire Mekong river, the banks were remarkably steep. According to this procedure we obtain the cross-section indicated as "proposed cross-section" in Figure 17. However, the cross-section might contain sandbanks or rock-outcrops which evidently do not have depth soundings when submerged. The "actual cross-section" can therefore be different than the proposed one, see Figure 17. If this is not recognized, a significant loss of wetted area can be the result of the order of magnitude of 10 to 20 percent which will influence considerably the obtained area-averaged velocity.

Figure 16 Example bathymetric map

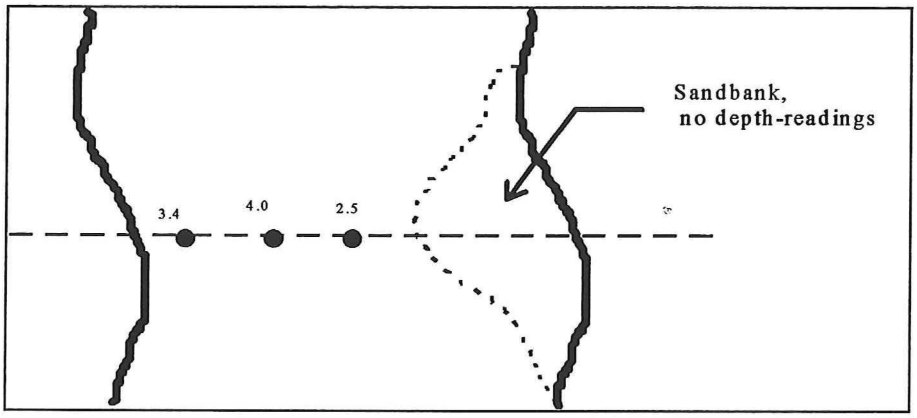
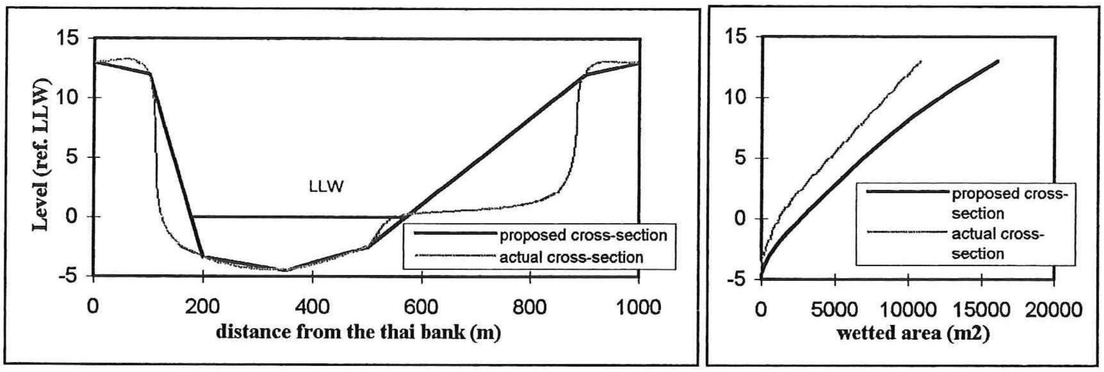


Figure 17 Actual and proposed cross-section and wetted area as a function of waterlevel



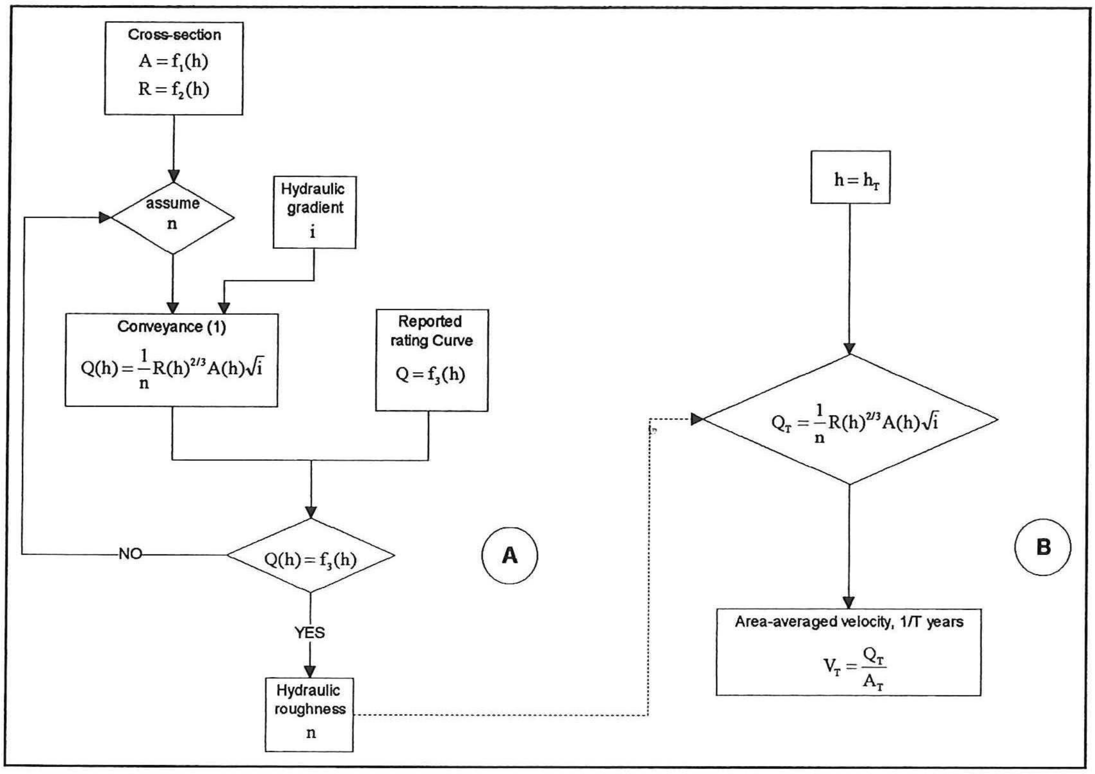
An analysis of several cross-sections indicated that the accuracy of the measured cross-section can be of significant importance. It was the author's impression that the uncertainty of the cross-sections is easily regarded in studies as being less important than other parameters like the hydraulic roughness. Part of this assumption might be based on the reluctance of doing time-consuming sensitivity analysis on these data.

16.2.3. Area-averaged velocity

The procedure to calculate the area-averaged velocities at the key stations for various return periods can be divided into two iterative steps, A and B.

A. The Manning coefficient, n, indicating the **hydraulic channel roughness,** is obtained by graphically matching the **reported rating curve** with the rating curve based on the measured **cross-sectional parameters.**
B. Once a reasonable reconciliation has been obtained, the **area-averaged velocity** is calculated from the discharges for selected return periods.

Figure 18 Procedure of deriving hydraulic roughness and area-averaged velocity



In the previous paragraphs it became clear that many of the elements in this procedure are subject to uncertainty which is not in all cases easily quantifiable. Especially in these cases a sensitivity analysis can be of great value as was explained before. In order to obtain as much efficiency and flexibility as possible for these analyses, all steps in Figure 18 should be integrated into one single program. In this study the spreadsheet program Microsoft Excel v5.0 was used as the program in which all elements were integrated. There are several reasons for this choice:

⇒ the program is very user-friendly
⇒ storing, preparing and editing of input and output of calculations is very easy
⇒ graphical- and non-graphical output is of high quality
⇒ object-oriented programming language Visual Basic is available
⇒ buttons, drop-down menus, dialog- and message boxes are easily programmable instruments to create interfaces
⇒ standardized analysis tools available like: what-if analysis, scenario-analysis

SPREADSHEET vs. PROGRAMMING

One of the most often heard objections against spreadsheet-programs brought forward by programmers who are used to work with languages like Pascal, FORTRAN or Basic, is that a model in such a program tends to become **unstructured** very easily. The **transparency** of the model is very low since a user can not directly see the structure of the model and the contents of the cells. It is difficult to prevent all users from making changes in the model and therefore the **integrity** of such a model is quite low.

Visual Basic, however, offers the possibility to use the sheets mainly as input-output data-sheets for separately programmed models. It is an object-oriented programming language, with a syntax that is much like Pascal. The object-orientation of the language allows the programmer to use and change all elements of the spreadsheet-program itself (and even other programs) very easily.

In Figure 19 the 'model-sheet' is shown of the cross-sectional calculations. This sheet consists of different fields of input and output data and two graphs, one of the cross-section and one of the rating curve. In this model the **input** data consist of:

⇒ general data of the cross-section
  * name
  * chainage
  * gauge datum

⇒ cross-sectional coordinates

⇒ reported rating curve

This input data is stored in a data-base. By choosing a certain cross-section from a pull-down menu, a program automatically gets the data mentioned above and puts it in the designated 'fields'[11].

Figure 19 Model - sheet "CROSCALC"

Station Nakhon Phanom
Chainage 1218 *km*
Gauge Datum 130.89 *m+MSL*

Cross-sectional data

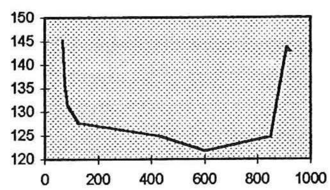
	x	y
1	67	145.2
2	75	136
3	80	134
4	85	131.5
5	125	127.7
6	200	127
7	425	125
8	600	121.8
9	850	124.8
10	910	143.5
11	920	143
12	925	142.8

Parameters

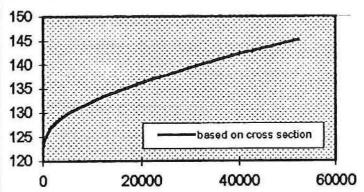
Hydraulic roughness
n = 0.02
Hydraulic gradient
i = 8.00E-05

CALCULATE Cross-sectional parameters

Cross-section



Rating Curve



Reported R.C. / ***Rating Table*** / ***Cross-sectional parameters***

	H	Q	Waterlevel	Discharge	Velocity	Konveyance	Area	Radius	Depth	Width
12	125.99	850	*121.8*	0	0.00	0.00E+00	0	0.00	0.00	0
13	126.89	1680	123.6	93	0.42	1.04E+04	224	0.90	0.90	248
14	127.89	2710	125.4	601	0.68	6.72E+04	885	1.87	1.87	472
15	128.89	3920	127.2	1718	0.89	1.92E+05	1921	2.83	2.83	679
16	129.89	5380	129	3835	1.18	4.29E+05	3241	4.30	4.31	752
17	130.89	7030	130.8	6768	1.47	7.57E+05	4617	5.93	5.94	777
18	131.89	8980	132.6	10427	1.73	1.17E+06	6033	7.60	7.61	792
19	132.89	11200	134.4	14752	1.98	1.65E+06	7467	9.29	9.31	802
20	133.89	13600	136.2	19666	2.20	2.20E+06	8920	10.95	10.99	812
21	134.89	16300	138	25177	2.42	2.81E+06	10387	12.62	12.68	819
22	135.89	19200	139.8	31227	2.63	3.49E+06	11868	14.27	14.36	826
23	136.89	22400	141.6	37795	2.83	4.23E+06	13362	15.91	16.03	834
24	137.89	25800	143.4	44433	2.99	4.97E+06	14874	17.26	17.41	854
25	138.89	29400	145.2	52187	3.18	5.83E+06	16417	18.95	19.13	858

[11] one method of assigning data to certain field in Visual Basic is by naming the ranges of cells and refering to them in the programs as an object.

The **output** can be generated by pushing the button "CALCULATE Cross-sectional data" that runs the program "CROSCALC". This is a program, stored in a separate program-sheet (see annex B, figure 4), which remains uncompiled and can be activated from different sheets. The output data consist of:

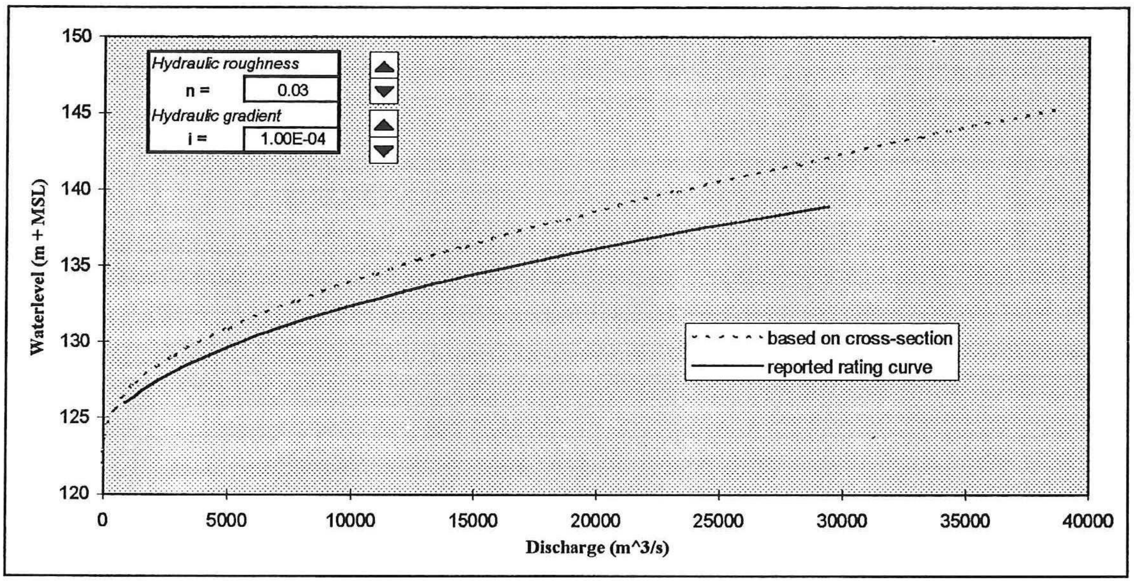
⇒ cross-sectional parameters, calculated by "CROSCALC" for 14 equi-distantial waterlevels over the total 'height' of the cross-section. These cells do not contain any formulas, they are simply the output-field of the program.
  * wetted area
  * radius
  * width at waterlevel
  * average depth

⇒ hydraulic parameters, which are calculated with the hydraulic roughness and gradient. These cells do contain formulas and are automatically updated when the hydraulic gradient or hydraulic roughness is changed
  * conveyance
  * discharge
  * area-averaged velocity

In this model the hydraulic gradient and hydraulic roughness are the two main calibration-parameters and therefore they are partly input-, partly output-parameters. In Figure 20 the rating curve of the cross-section is shown together with the reported rating curve. The rating curve based on the cross-section can be changed by varying one of the two parameters with the so-called 'spinners', the up-down buttons with which the value of a target cell can be varied by pre-defined steps within pre-defined ranges. This change is directly visible in the model-sheet and in the rating-curve graph itself.

A possible range of the derived variable (the hydraulic roughness) can be derived for which the rating curve based on the cross-section can still be agreeably matched with the reported one, taking the uncertainties of the observed parameters into account (the cross-section and the hydraulic slope). Although the decision of what is a good match has to be taken subjectively by the engineer, this still is a better way than deriving one 'design value' from one curve that matches, which is even more arbitrarily. If this parameter is used in further calculation, the PDF of the hydraulic roughness can be assumed to be, for example, a uniform distribution over this range if the match was equally well over the whole range of n, or a normal distribution if the goodness of fit decreased when deviating the roughness from a mean value.

Figure 20 Visual fitting of rating curves, varying i and n



The choice between programming a certain routine calculation in Visual Basic or using formulas in cells is based on the frequency that this data has to be changed. Cross-section coordinates should be easily edited to allow sensitivity analysis. Also for several cross-sections more detailed soundings became available during the study and other cross-sections appeared to have been misinterpreted from the bathymetric map and had to be adjusted. However, this doesn't mean that they have to be edited constantly. When calibrating the hydraulic roughness and hydraulic gradient to match the reported rating curve the hydraulic parameters will have to be recalculated constantly with every variation.

16.2.4. Near-bank velocity

Under the auspices of the National Energy Authority (NEA), velocity measurement campaigns have been executed in several years. For design purposes, only the velocity measurements in 1971, a high flow year, are of interest. In this year, measurements were carried out during 54 days, from January to December. These measurements can be used to verify the area-averaged velocity obtained with "CROSCALC".

The depth-averaged velocity in a vertical was derived as the average of the velocities measured at 0.2 and 0.8 times the depth in each individual sub-section or at 0.6 times the depth for shallow verticals (in accordance with the method described in French (1985)). The area-averaged velocity is defined as the weighted sum of the depth-averaged velocities, where each weighting factor is defined as the sub-sectional area divided by the total cross-sectional area. A typical graph showing the velocity measurements and the depth- and area-averaged velocities at Nong Khai on August 20, 1971 is shown in Annex B, Figure 5 and 6. In Annex B, Figure 7 a plot of these ratios over the year at 60 m, 90 m and 120 m from the right bank is presented. From the graph it can be seen that a ratio of 1.1 is a reasonable first guess. As explained before, for the actual design velocity also planform and geometry of the cross-sections will have to be studied in detail for the considered sites.

BEHIND THE DATA

During the analysis of the velocity measurements it was found that the measurements were taken at different cross-sections around km 1550. This managed to explain sudden changes in the form of the cross-section. It also appeared that velocity measurements were done with different instruments for different days of the year ('NEYRPIC' and 'Hilger-Watts'). This managed to explain sudden changes in velocities from one day to the next. The depth measurements, to determine at what depth to measure the velocity, were done with a cable with a weight, making the stated accuracy of the reported depth, in three digits, questionable. After a thorough study of the available data, some datasets were decided to be taken out of the analysis. This clearly illustrates that even in case of large quantities of data, it is not at all obvious to use all this data directly in the analysis. Data validation and determining homogeneity are the essential first steps.

16.3. Morphological parameters

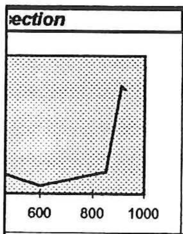
In the morphological study, the cross-sectional data (hydraulic radius, wetted area, etc.) were needed as well, to calculate several morphological parameters, see Table 3. These parameters had to be derived for many cross-sections, for discharges with certain return periods and different properties of the river bed sediment. During the analysis, different cross-sections became important for the analysis and other return periods appeared to be more determining for certain morphological processes than the ones previously used. These types of changes resulted in the necessity of repeating many of the calculations.

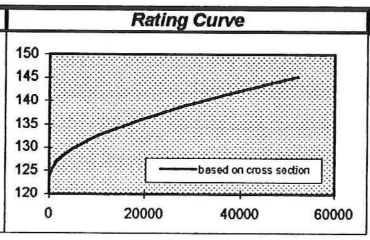
To provide the same flexibility as mentioned before, a small program, named "MORPHO", was developed as an extension of "CROSCALC". This routine calculates the value of several of the morphological parameters, for various waterlevels. The output of these calculations are put in another part of the same model-sheet as "CROSCALC", see Figure 21. Several other input parameters are needed to do the calculations:

- ⇒ density of the sediment
- ⇒ median size of the sediment particles
- ⇒ exponent, b, from the sediment transport formula $s = m\,u^b$

The results can be generated by pushing the button "CALCULATE Morphological parameters" that runs the program "MORPHO". In annex B, figure 8, the source code of this program can be found.

Figure 21 Model-sheet "MORPHO"





CALCULATE Morphological parameters

Sediment properties		
density	ρs	2600
permittivity	ks	
porosity	n	
	D10	
	D50	3.00E-04
	D90	
transport exponent, b		3

		Cross-sectional parameters					Morphological parameters				
Velocity	Konveyance	Area	Radius	Depth	Width	Levels	Shields	Suspension	Interaction	Lp	LD
0.00	0.00E+00	0	0.00	0.00	0	121.8	0.00	0.00	0.00	0	0
0.42	1.04E+04	224	0.90	0.90	248	123.6	0.19	0.39	0.04	3095	5115
0.68	6.72E+04	885	1.87	1.87	472	125.4	0.39	0.56	0.04	7991	12791
0.89	1.92E+05	1921	2.83	2.83	679	127.2	0.59	0.69	0.04	13656	21564
1.18	4.29E+05	3241	4.30	4.31	752	129.0	0.90	0.85	0.07	18305	21438
1.47	7.57E+05	4617	5.93	5.94	777	130.8	1.24	1.00	0.12	22186	19478
1.73	1.17E+06	6033	7.60	7.61	792	132.6	1.58	1.13	0.18	25969	17900
1.98	1.65E+06	7467	9.29	9.31	802	134.4	1.93	1.25	0.25	29910	16587
2.20	2.20E+06	8920	10.95	10.99	812	136.2	2.28	1.36	0.33	34284	15658
2.42	2.81E+06	10387	12.62	12.68	819	138.0	2.63	1.46	0.42	39424	14849
2.63	3.49E+06	11868	14.27	14.36	826	139.8	2.97	1.55	0.51	45821	14214
2.83	4.23E+06	13362	15.91	16.03	834	141.6	3.31	1.64	0.61	54387	13704
2.99	4.97E+06	14874	17.26	17.41	854	143.4	3.60	1.71	0.68	63333	13803
3.18	5.83E+06	16417	18.95	19.13	858	145.2	3.95	1.79	0.80	83642	13295

Table 3 Morphological parameters

Shields parameter	θ	Parameter indicating stability of sediment against flow. This parameter is defined as: $\theta = \frac{hi}{\Delta D}$
Particle shear velocity	u*	This parameter is defined as: $u = \sqrt{ghi}$
Particle fall velocity	w	This parameter is defined as: $w = 1{,}1\sqrt{\Delta gD}$ for D > 1 mm and $w = \frac{10\upsilon}{D}\left\{\sqrt{1+\frac{0.01\Delta gD^3}{\upsilon^2}} - 1\right\}$ for 1 mm > D > 100μm
Suspension parameter	u*/w	Parameter in which the particle shear velocity u*, is divided by the particle fall velocity, w. It defines the threshold for suspension and non-suspension. The particles for which u*/w > 1, are in suspension.
Relaxation length of the sediment movement	λ_s	This parameter indicates the characteristic length needed for the river to adjust its bedform to the equilibrium bedform for about 67%, after a 'disturbance' in this bedform, such as a bend. It is defined as: $\lambda_s = \frac{h}{\pi^2}\left(\frac{B}{h}\right)^2 0.85\sqrt{\theta}$
Relaxation length of the water movement	λ_w	This parameter is similar to the parameter above, it indicates the characteristic length needed to reach 67% of the equilibrium profile of the velocity of the water after a disturbance. It is defined as: $\lambda_w = \frac{C^2 h}{2g}$
Interaction parameter	λ_s/λ_w	This parameter can give a indication of the number of channels that are likely to exist in the river planform.
Damping coefficient	$1/L_d$	This parameter is defined as $\frac{1}{L_D} = \frac{1}{2\lambda_w}\left(\frac{\lambda_w}{\lambda_s} - \left(\frac{b-3}{2}\right)\right)$
Wave number	$2\pi/L_p$	This parameter is defined as $\frac{2\pi}{L_p} = \frac{1}{2\lambda_w}\sqrt{(b+1)\left(\frac{\lambda_w}{\lambda_s} - \left(\frac{\lambda_w}{\lambda_s}\right)^2 - \left(\frac{b-3}{2}\right)^2\right)}$

The last two parameters come from the equation which describes the progression of waterdepth disturbances in a river:

$$H(x) = H(0)\exp\left(-\frac{x}{L_D}\right)\sin\left(\frac{2\pi}{Lp}(x + x_p)\right)$$

H(x) and H(0) are the amplitudes of the waterdepth disturbances at x = x and x=0 and x_p is the spatial lag of the harmonic movement.

note: in morphological analysis, the parameter C (the Chezy coefficient), is often used as roughness coefficient. This parameter can be derived from the Manning coefficient of roughness, n, through the relationship: $C = \frac{R^{1/6}}{n}$

17. Data management in the analysis process

In the Mekong River Bank Erosion Study, a long stretch of river had to be characterized (>900 km) and a large amount of data was available and produced during this study. To analyze all this data, a large variety of computer programs is used to perform certain specific tasks. This ranges from programs that perform small tasks, like data-checking, data-fitting, calculating cross-sectional parameters and graphical presentation to programs that perform complicated and time-consuming numerical calculations, like some hydrological and morphological models. The latter kind of models will not be discussed here, the focus will be on the group of relatively small programs that support the engineer in analysis and design.

In the analysis and design process, the output of one program is often the input for another. Although all these programs have been designed specifically for their task and can proof to be very functional, they also result in laborious input-output conversions and occasional compatibility problems. Because of these problems, this group of programs does not allow for easy updating if input, boundary conditions or formulas change. This would mean redoing the same series of small 'batch processes' which is not a very welcome activity.

Probabilistic analysis, and especially sensitivity analysis, is likely to be one of the first aspects to get less attention because of these 'information-logistical' problems. First of all, because it is an extra burden on the time-schedule and secondly because the input-output conversions require the communication of uncertainty between different software programs and different computers. This appears to be done in a very limited way, probably because of lack of convention or agreement on how to perform this communication.

17.1. Database-calculation requirements

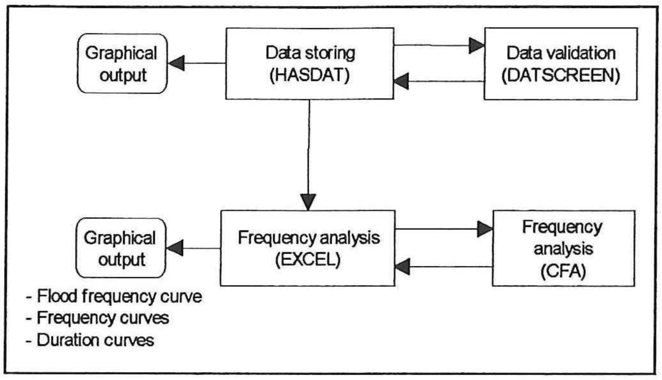
One of the most difficult aspects of 'data-management' is the link between data-storing and data-calculation. For data-storing, the emphasis has always been on providing the consistency and integrity of the data stored, because this data should not be edited very frequently. This has led to development of databases, most of which have a relatively 'closed' structure. In data-calculation programs like spreadsheet-programs, the structure is extremely 'open' and allows for almost unlimited editing of the data. In the analysis process both aspects are useful for different types of data.

17.1.1. Observed parameters

As was stated before, the observed parameters are the parameters of which values have been measured and therefore reflect the state of nature at a certain moment. These parameters should be stored in a database with relatively high integrity, because they are not meant to be changed unless errors are found. The access to the database in order to retrieve data should be very easy, however, because it is used in several analyses. In the MRBES, this proved to be rather time-consuming because of the large amount of available data.

For the hydrological analysis, discussed in Chapter 15: Uncertainty of 'observed parameters', the waterlevels and discharges were stored in the database HASDAT. To analyze this data, the files had to be retrieved from this database in ASCII-format and imported in the program to perform the analysis with. The data flow is illustrated in Figure 22.

Figure 22 Data flow hydrological analysis MRBES

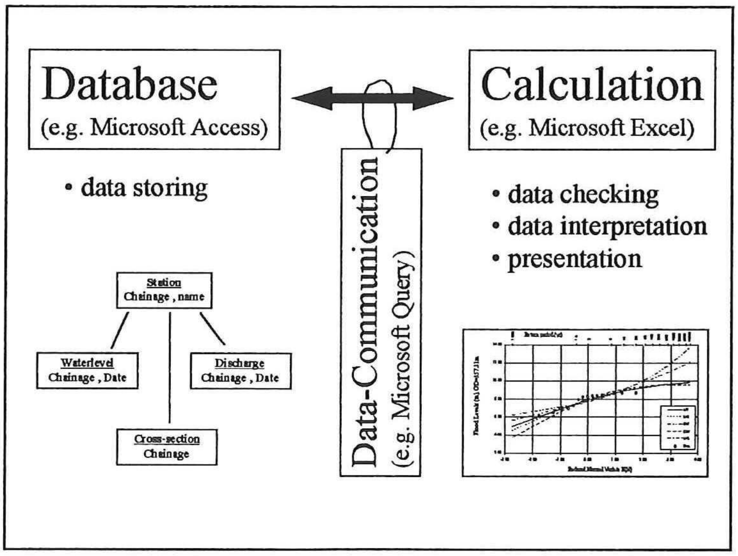


Basically, there are two ways to make this data-flow more efficient en flexible:

1. The first alternative would be to integrate the data-evaluation programs into the database and create one single program because this would prevent the conversion- and the compatibility problems. The main problem with this strategy is that this program might become very big and difficult to maintain. Furthermore there is the problem of choosing the programs which should be integrated because it will be impossible to include all useful programs.

2. The second alternative is to improve the interface between the data-storing program and the data-evaluation/calculation programs. The latter kind of programs should have direct access to the database-files and select the relevant data without having to convert it. This alternative is the idea behind a program called Microsoft Query. With this program, the user can 'ask questions' at a database. In other words the user can select data from a database and retrieve this data directly in, for example, a spreadsheet where the data could be the input for a program or the points of a graph. In this way the functionality of both programs can be exploited without having to use one big program with a lot of overhead. This strategy is shown in Figure 23.

Figure 23 Proposed storing-calculation system for observed parameters

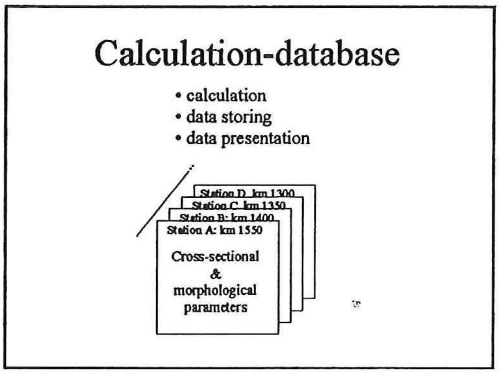


17.1.2. Derived parameters

Derived variables are parameters that are calculated based on observed or assumed parameters. These parameters should be easy to recalculate for different conditions and therefore the structure of the program that performs these calculations should be, unlike observed-parameters, as open as possible.

In a previous paragraph, several hydraulic and morphologic parameters were calculated for different waterlevels in one cross-section. This is a one-dimensional analysis of these parameters based on a two-dimensional definition of the cross-section. A two-dimensional analysis of these parameters can be made if these cross-sections are put in a series 'behind' eachother. Now, for every waterlevel, a longitudinal overview can be made for each parameter, a kind of 'fingerprint' of the river. The most recent releases of spreadsheet-programs are all based on three-dimensional spreadsheets in which this can be done simply by connecting the corresponding cells of all sheets.

Figure 24 Proposed storing-calculation system for derived parameters



Many of the observed parameters, however, were available for different sets of cross-sections. The riverwidth was available every 1 km (measured from the bathymetric map), the first set of complete cross-sections every 50 km and discharges at the eight main hydrological stations along the river. If parameters have to be calculated using input parameters with incongruent sets of cross-sections, it is necessary to interpolate between known values or to enter new data. In order to make graphs it might be necessary to combine observed-parameters and derived parameters.

The most straightforward way to deal with this problem is to create a database in a spreadsheet placing every separate value of a parameter in a cell and every separate parameter in a different column. The difference between two consecutive cells in every column will have to be the smallest spatial increment of the longitudinal overview, for example 1 km. This will create large spreadsheet-files and the calculations between columns will slow the program down enormously. Also, because the calculations performed in the spreadsheet will involve formulas in cells, the previously mentioned lack of transparency and integrity in such a model will create problems very easily. The advantages are that all values are updated automatically and that every value is in a separate cell which is a requirement for making graphs.

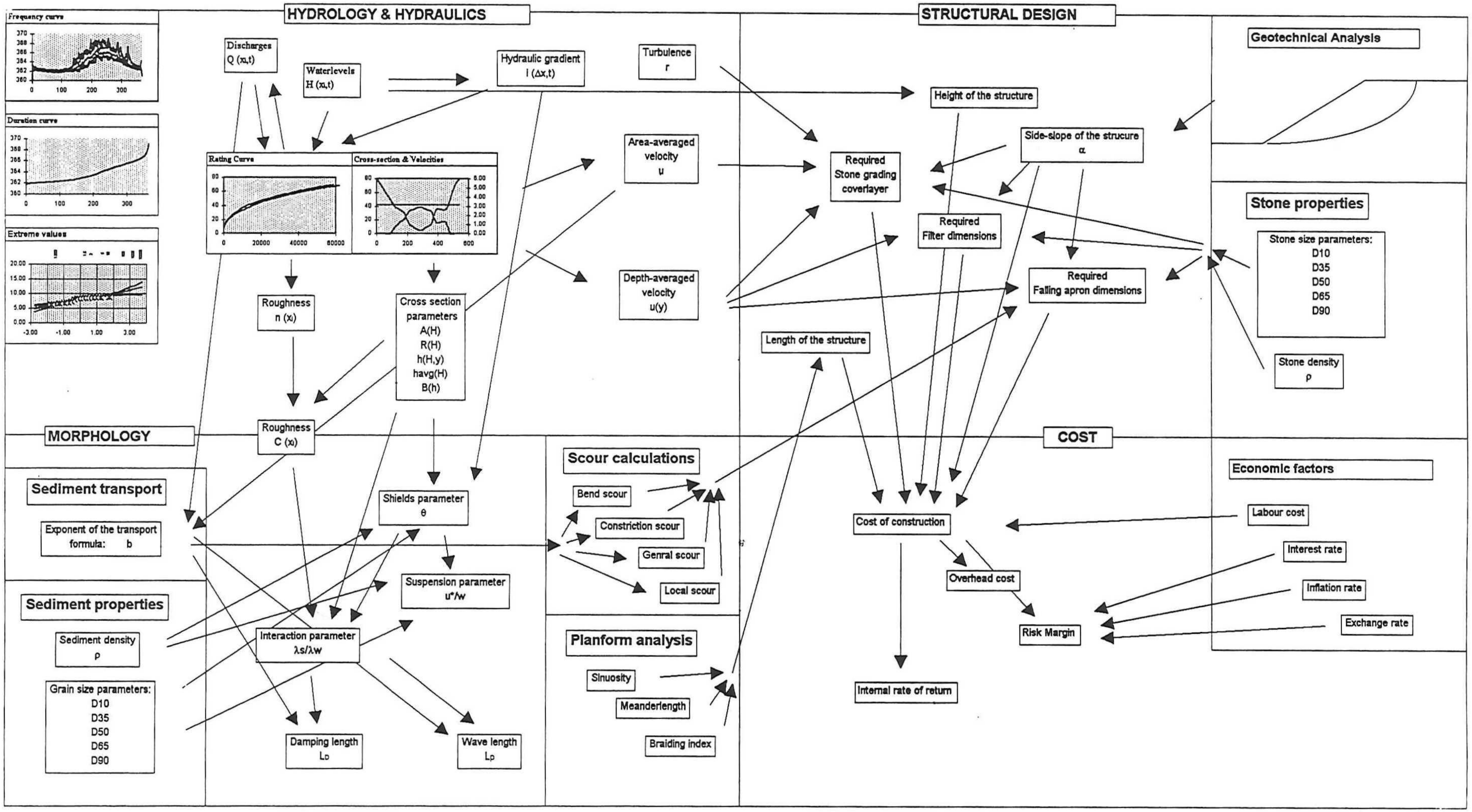


Figure 25 Parameter flow diagram for the MRBES

17.2. Communicating uncertainty

17.2.1. Parameter flow

In the analysis and design phase of studies, parameters and variables are introduced or taken from other partial studies completed in a previous project phase. As discussed before, all these parameters have a certain degree of uncertainty. This uncertainty is not a permanent attribute of the parameter. As these parameters make their voyage through all the study phases, as an observed parameter or an auxiliary parameter being transformed into other parameters, they acquire more uncertainty or become more certain. For example, if measurements confirm an earlier estimate or previous measurements, its uncertainty will decrease. If a new parameter is introduced as a function of several others, it will acquire the composite uncertainty of these parameters.

As in the MRBES, it might sometimes not be fully recognized or identified how all the parameters are 'traveling' through the design process and who is using, comparing or deriving which particular parameter. This can result in engineers independently deriving the same parameters, sometimes even from the same data. This is the result of many factors of which computer-software and project-organizational aspects are two of the most important.

Yet, for a full and consistent probabilistic approach in the design it is necessary that the data-flow is identified. A concise way to do this would be a **parameter flow diagram**. As an example, a parameter flow diagram of the MRBES was established (see Figure 25). In this figure, the total project is split up in partial studies. In such a diagram it can be easily identified which parameters cross-over from one study to another. These are the 'nodes' where the parameters will have to be communicated. This is especially important if the borders between the studies are indeed physical borders where the data goes from one computer to the other or from one program to the other.

If such a Parameter Flow Diagram is established at the start of the project and is adjusted as the project goes on, this can increase significantly the efficiency, consistency and maybe even the quality of analysis within the project.

17.2.2. Reliability label

At several points in the parameter flow diagram certain parameters have to be communicated between persons, computers or programs. The most classic way in which to do this is with design values: one single value representing the parameter. This is often the mean value or an extreme value with a certain return period or probability of exceedence. However, in deriving or analyzing this parameter in a partial study, often much more information about the parameter's uncertainty has been derived. In applying only one single value for parameters, a lot of this information will not be communicated. Still some of this information will be needed in a later phase of the design process, when sensitivity analyses and probabilistic calculations are performed and the progression of errors is discussed.

A more concise way to reflect a parameter's uncertainty is with a **Probability Density Function**. The parameter is characterized by a distribution type and its distribution-parameters. For example NORMAL(μ, σ) or GUMBEL(α, β). But as was discussed in the first section, the choice of the model and deriving its parameters are also subject to uncertainty (model and statistical uncertainty). An engineer who has to use the parameter might want to know how good a certain distribution fitted the observation. Part of this uncertainty can be captured by stating the confidence intervals on the distribution's parameters. Furthermore the available observations could be shown in a **Probability Mass Function** (histogram) with sample moments. If no observations are available, other sources of information could be communicated like **confidence statements.**

Not all levels of detail will be exactly relevant to the user(s). It is important, however, that a certain level is indeed agreed on in order to make it a standard procedure in the design methodology. A possible way to do this is with a **Reliability Label** on which the essential information about the parameter could be summarized upto the necessary degree of detail.
The main advantages of using such a label would be:

1. Being able at any moment of time in the design process to answer questions about parameter uncertainty (can be useful because the output of some engineers may be halfway the total design process).
2. It forces engineers to consider uncertainty and makes them more oriented towards minimizing it.

> ***KANBAN***
> A reliability label can be compared with the concept of 'Kanban', a well-known logistic method in the manufacturing industry, developed at the Japanese car manufacturer Toyota in the seventies. Kanban simply means 'fiche' or' label' and embodies the idea that each physical item in a manufacturing process should have one single label on which its specifications are summarized. During the manufacturing process the contents of this label can be changed but is at every given time unique; there are no other specifications for the same item at the same time.

The following general lay-out could be suggested for such a label, see Figure 26:

⇒ General information

⇒ Statistical information

a) Probability Density Function

b) Sample information (if available)

⇒ sample histogram

⇒ moments of the sample

⇒ confidence interval on PDF. parameters

c) Confidence statements with reference & Interpretation of these statements

Figure 26 Reliability Label

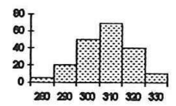
1 Parameter description

D_{50}, stone diameter for which 50% of the stones (in weight) in the total grading are smaller.

2 Probability Function type (assumed)

$$f(x) = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2}$$

3 Sample statistics & fitting results



$n = 200$

$\mu = 300$

$\sigma = 10$

$\chi^2 = 0.225$

4 Confidence limits of the function's parameters

the α % confidence limits on μ are $\bar{x} \pm \frac{\sigma k_{\alpha/2}}{\sqrt{n}}$

Additional information

Source of the parameter

Confidence statements

It is acknowledged that the Reliability Label is still a rather abstract notion in this study. The way in which to apply it practically is not fully clear. It might not be practical to do this for every parameter, but for some key parameters it might prove to be useful. The essential suggestion in this paragraph is to agree on how to communicate uncertainty in a certain project and to propose which type of information could be useful to include.

Histogram & Sample moments

If data has been checked and accepted, we can use the probability mass function (histogram) as an approximation of the probability density function and derive the sample moments (see Annex B, BOX 3). It should be recognized, however, that the sample moments are just estimates of the real values, subject to uncertainty which should be recognized and, if possible, quantified. The fact that the data was accepted after significance testing doesn't mean this uncertainty has been taken away.

Confidence intervals

The confidence interval provides an interval estimate on a parameter instead of a point estimate. When the distribution is a part of a larger probabilistic model engineers seldom afford themselves the luxury of taking into account more than a point estimate. But this interval is in fact very useful as it emphasizes that the value of the parameter is not precisely known and that there is a certain magnitude of uncertainty involved. The use of confidence limits (see Annex B, BOX 4) is very appropriate in the reporting of data. They are convenient to use and widely understood.

Confidence statements.

In many handbooks and scientific publications, statements are made about the uncertainty of parameters or models. These kind of statements will be called "confidence statements". In general, we will have many different types of confidence statements from different sources, in different 'formats' and, of course, with different credibility.

Although these statements often contain useful information, this is not always recognized or, if recognized, the user might not know how to interpret or translate it. This is most likely the result of a lack of convention in communication and notation of uncertainty.

Some of the types of confidence statements encountered during the MRBES study are:

- Typical ranges of parameters for different conditions: $x \in [x_1,x_2]$ | Conditions
- The probability of (non-) exceedence of a certain value: $P\{x>x_1\}= p_1$ | Conditions
- The limit of the standard deviation: $\sigma < \sigma_{max}$ | Conditions
- Limit of the coefficient of variation (CV): $CV < CV_{max}$ | Conditions
- Confidence limits: $P\{x_{upperlimit} < x < x_{lowerlimit}\} = p$

Some examples of these confidence statements encountered are cited in Annex B, BOX 5.

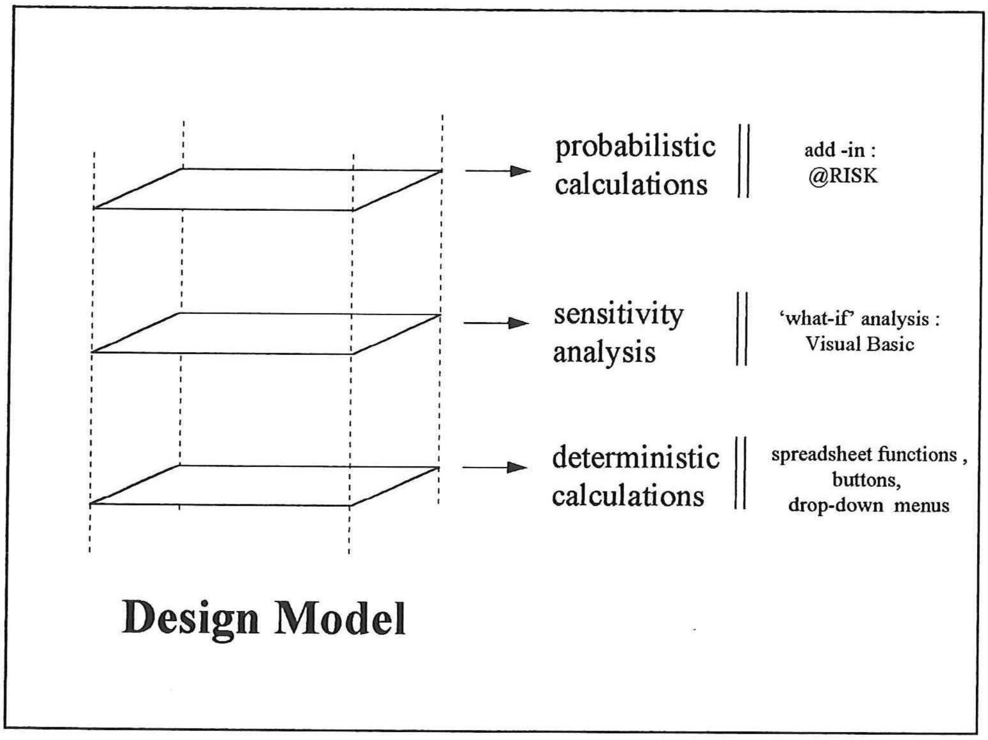
18. Design Model

The design phase consists of many different aspects and considerations. For all these aspects many small computer programs are available that were developed by HASKONING or other companies or institutes. Also various different spreadsheet-programs are used to do straightforward deterministic calculations. Probabilistic calculations require a separate program. In the MRBES the program HASPROB was used (proprietary software HASKONING) which is based on the level II method of probabilistic calculation AFDA.

The situation described above leads to the observation that some of these calculation models are redesigned several times in different programs. A model will be developed in one or more programs for deterministic calculations and graphical output and has to be reconstructed in another program for the probabilistic calculations. Ofcourse it would be more efficient if all the aspects of the design would be integrated in one program, because many parameters are used in more than just one aspect of the design. Furthermore, it would be more efficient if the 'infrastructure' of this model, the functional relationships between the parameters, could serve for both deterministic and probabilistic calculations.

The aim is the integration of different levels of detail of probabilistic analysis into one single model. This idea is illustrated in 26. To achieve this, use was made of two computer-programs that have been developed during the last few years. The 'skeleton' of the model is spreadsheet-program Microsoft Excel v5.0 and the main 'limb' is the add-in @RISK, developed by Palisade Co.. An add-in is a program that can be attached to the main spreadsheet-program to perform specific tasks. The specific task of @RISK is doing risk-analysis by running simulations with the Monte Carlo method of sampling (see section I).

Figure 27 Single Design Model with different levels of risk-analysis



18.1. Deterministic calculations

One of the most important reasons to perform deterministic calculations is to make a quick assessment of the possible design types and their required design dimensions and cost. This allows for a pre-selection of promising and less promising alternatives. For the promising alternatives, a preliminary design can be made deterministically. In this paragraph a design-support model for the preliminary design of the bank protection is discussed.

Like in the studies discussed in section II, the preliminary design was mainly based on the following design criteria:

1. current stability
2. wave stability
3. stability for expected extreme scour

This design-support-model was set up in the spreadsheet-program Excel v5.0. The handbook from PIANC: "Guidelines for the design and construction of flexible revetments.."(1992) was used as reference for this model. The criteria mentioned above were all put in separate 'sheets' of a 'workbook'. In every sheet, the designer treats a different aspect of the design.

The most important objective of this design model is that the user should be very flexible in evaluating different design scenarios and in testing the sensitivity of certain parameters. Furthermore the designer shouldn't have to refer to a guidebook at any time[12]; all the information that is needed should be in the model. This is obtained by using the following tools:

- All the important **equations** are shown in boxes on the sheets
- **Tabulated factors and parameters** can be chosen from drop-down menus or manually, like the turbulence factor in the current stability-sheet, see Figure 28.
- Some of the other parameters can be varied by pre-defined steps within pre-defined ranges with so-called 'scroll bars'
- Colors are used as a guidance for the user. Values that can be edited are marked in green (in the figures these cells have a gray background), the dependent cells are marked in red and also 'locked' to prevent editing
- **Messages** that can be informative for the user are shown in cells, like the indication if we are dealing with surging- or plunging waves in the wave stability-sheet, see Figure 29.
- Simple **graphs** allow for a visual check of the calculations, see Figure 30.

[12] taking along all the necessary literature is a well-known burden for consultancies operating in foreign countries

Figure 28 Current stability - design sheet

Current stability according to Pilarczyk (1990)

$$\Delta_m D_n = \Phi_s K_t \frac{0.035}{\Psi_{cr}} K_h K_s^{-1} \frac{u^2}{2g}$$

stone density	ρ	2650 [kg/m3]	
Relative density	Δ	1.65 [-]	
Stability factor	Φs	0.75 [-]	continuous protection of loose units
Turbulence factor	Kt	1.81 [-]	relative turbulence intensity > > > 25 %
Critical shear stress	Ψcr	0.035 [-]	rock
waterdepth	h	15 [m]	
Depth factor	Kh	0.46 [-]	undeveloped flow
slope (1 : n)	n	3 [m]	
internal friction	ϕ	35 [°]	
Slope factor	Ks	0.83 [-]	
Current velocity	u	2.8 [m/s]	
Stone diameter	Dn	0.18 [m]	
Stone diameter	D50	**0.21 [m]**	

Figure 29 Wave stability - design sheet

Wave stability according to Pilarczyk (1990)

$$\Delta_m D_n = \left(\Phi_s \Psi_u \cos(\alpha)\right)^{-1} \xi_m^b H_s$$

stone density	ρ	2650 [kg/m3]	
Relative density	Δ	1.65 [-]	
accepted damage level	Sd	7 [-]	service limit state
number of waves	N	3000 [-]	
permeability factor	P	0.1 [-]	armour, filter, impermeable core
Stability factor	Φs	2.71 [-]	$\Phi_s = 6.2\, P^{0.18} \left(S^2/N\right)^{0.1}$
Upgrading factor	Ψcr	1 [-]	Riprap
slope (1:n)	n	2.5 [m]	
mean wave period	Tm	2 [s]	
Breaking index	ξm	1.84 [-]	plunging waves
interaction exponent	b	0.50 [-]	
Significant wave height	Hs	0.25 [m]	$\xi_m = \frac{\tan(\alpha)}{\sqrt{H_s 2\pi / gT^2}}$
Stone diameter	Dn	0.13 [m]	
Stone diameter	D50	**0.16 [m]**	

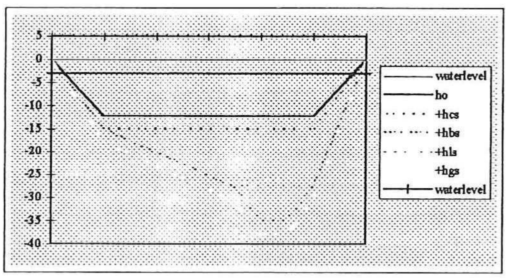
Wave forecasting according to Bretschneider (1977)

Fetch length	F	1000 [m]
Wind speed	u	10 [m/s]
Waterdepth	h	10 [m]
Wave height	Hs	0.25 [m]
Wave period	T	2 [s]
Wave length	L	5 [m]
Dimensionless parameters		
Wave height'	H'	0.024 [-]
Period'	T'	1.79 [-]
Fetch'	F'	98 [-]

Figure 30 Scour calculation - sheet

Expected scour

			model factor, c.s.	k1	0.8 [-]
Initial waterdepth	ho	12.2 [m]	*model factor, b.s.*	k2	1.83 [-]
			model factor, l.s.	k3	0.3 [-]
+Constriction scour	hcs	15 [m]	Constriction scour	Δhcs	2.8 [m]
+Bend scour	hbs	27.45 [m]	Bend scour	Δhbs	12.5 [m]
+Local scour (30yrs)	hls	34.85 [m]	Local scour (30yrs)	Δhls	7.4 [m]
+General scour (yr)	hgs	37.85 [m]	General scour (yr)	Δhgs	3 [m]
Total waterdepth	hts	37.85 [m]	Total scour	Δhts	25.6 [m]

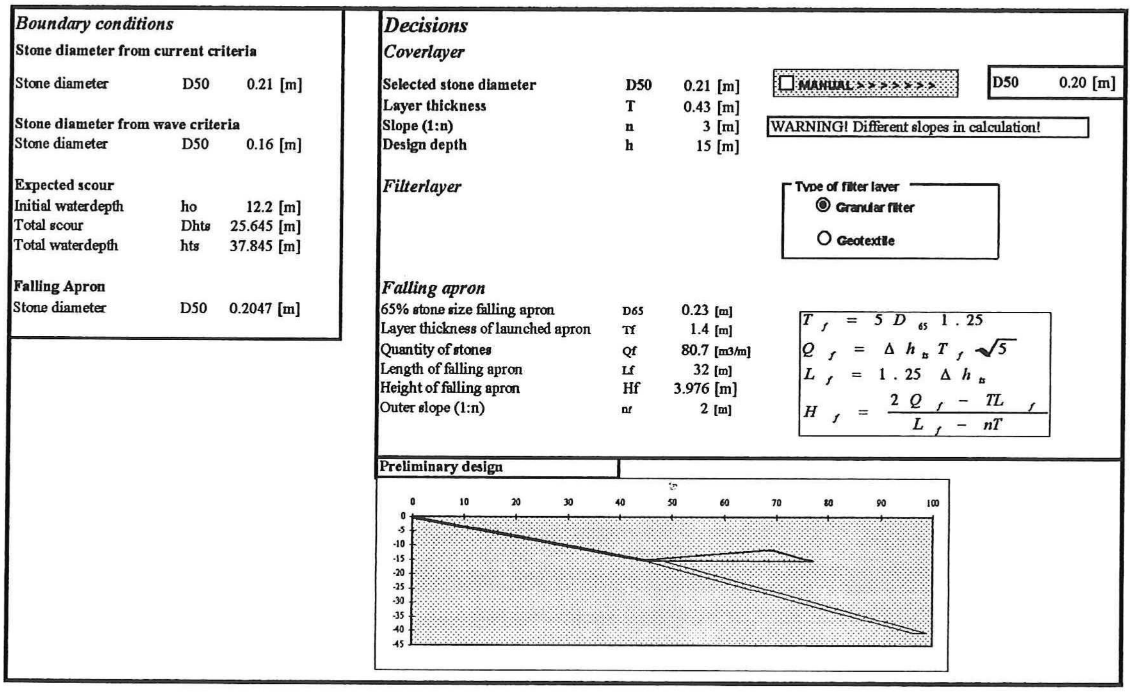


$$h_{cs} = k_1 h_o$$
$$h_{bs+cs} = k_2 h_{cs}$$
$$\Delta h_{ls} = k_3 h_{bs+cs}$$
$$h_{ts} = h_{bs+cs} + \Delta h_{ls} + \Delta h_{gs} = k_1 k_2 (1 + k_3) h_o + \Delta h_{gs}$$

Finally all the results of the various design aspects are summarized in a 'decision-sheet', see Figure 31. The different sheets will be checked on consistency in the decision sheet. If, for example, the designer uses slope 1:3 for the current stability calculation and 1:3.5 in the wave stability calculation, this will appear as a warning in the decision sheet. It is not a problem to have different conditions for the different criteria while trying different scenarios in one sheet, but when everything has to be summarized in one single design, the design should be consistent.

In the decision-sheet the user can decide whether the preliminary design will be calculated with the calculated minimal required dimensions or if the user wants to change these values manually. The final decisions are still to be made by the user because the model should not be a black box.

Figure 31 Decision design - sheet



18.2. Probabilistic calculations

In the previous paragraph the deterministic design-model was presented. In such a model only one design-scenario at a time was calculated. The values of parameters were either directly put in a cell or chosen from a drop-down menu.

To do a sensitivity analysis in this model, different scenarios can be studied by changing the input parameters and saving every scenario with the "scenario-manager". Furthermore, a "what-if" analysis can be done with the table-option. This is an automatically updating table with either:

1. a value-range for one input parameter against several dependent output parameters
2. value ranges of two changing parameters against one output parameter

With sensitivity analysis we can not determine any probabilities of failure, however. Therefore we need to do detailed probabilistic calculations. To perform these calculations in the described models, we need the add-in program @RISK (Palisade Co.) which can be connected to the spreadsheet program Excel[13]. This add-in creates the possibility to define a probability distribution, instead of a value, and enter this function in a single cell. This probability distribution looks like a normal spreadsheet-function but is used in a different way. A normal distribution will be "= @NORMAL(μ, σ)" and a Weibull distribution will be "= @WEIBULL(α, β)".

In the straightforward spreadsheet-calculations, these functions return their expected value in the cell, with which can be calculated as with normal values. If the add-in is activated, a simulation can be executed. During a 'simulation run' the program performs a so-called 'What-if analysis': for a specified number of iterations, samples are taken from all PDF's defined in the spreadsheet, and each iteration the spreadsheet is recalculated and the results are stored. After the simulation run, the results of specified output cells can be displayed as frequency histograms. These frequency histograms will approximate the PDF of the parameter in that cell, the accuracy of this approximation depending on the number of iterations that is done.

As an example, the Shields-Chezy formula for current stability will be used to make the Z-function:

$$Z = \Delta D_n - \frac{K_h K_T}{K_s} \frac{2g/18^2}{\Psi_{cr}} \frac{u^2}{2g} \qquad \text{with} \qquad u = C\sqrt{hi}$$

and h is a function of the discharge Q.

The distribution functions used in the simulation are shown in Table 4. The uniform, discrete and normal distributions are available as standard functions in @RISK, the Gumbel distribution is constructed according to BOX 3. The results of one of the calculations are shown in Figure 32. The resulting graph of the Z-function of one of the three simulation runs is shown as well as the simulation statistics and part of the model as it appears on the spreadsheet. The return values are the expected values of the distributions.

Table 4 Example @RISK model

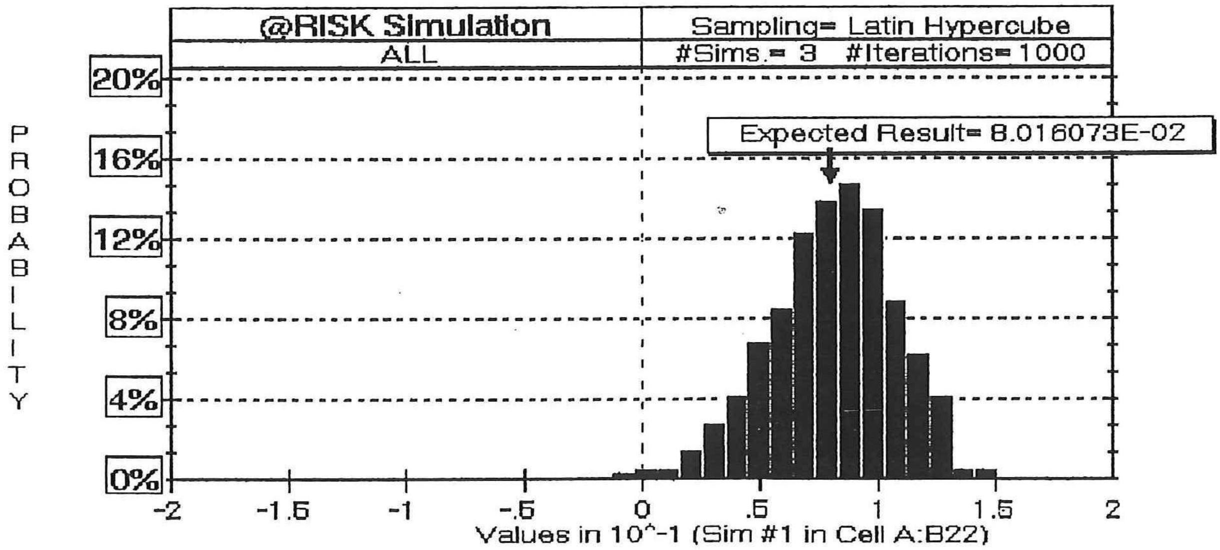
Parameter	Symbol	PDF
Stone density	ρ_s	@UNIFORM(2550, 2650)
turbulence	r	@DISCRETE(0.1, 0.5, 0.15, 0.3, 0.2, 0.2)
Shields stress	Ψcr	@NORMAL(0.04 ,0.004)
Slope protection	α	@NORMAL(16 ,1.6)
Friction angle	ϕ	@NORMAL(35 ,3.5)
Stone diameter	Dn	@NORMAL(0.1 ,0.01)
Discharge	Q	@GUMBEL(60862, 6603)
Roughness	C	@NORMAL(70 ,7)
Gradient	i	@NORMAL(2E-5 ,0.2E-5)

[13] @RISK is available for several releases of the spreadsheetprograms Lotus 123 and MS Excel. In this study only a release for Lotus v.3.1 for DOS was available. But because there are no fundemental differences with versions for Excel, the examples are with Excel.

Figure 32 Results probabilistic calculations @RISK

Current stability according to Shields-Chezy formula

stone density	2600	Discharge	13635.85
Relative density	1.6	Stage	23.18266
		Chezy	70
relative turbulence intensi	0.15	Gradient	2
Turbulence factor	1.244		
		Velocity	1.507283
Critical shear stress	0.04		
waterdepth			
Depthfactor	0.33		
slope of protection	16		
angle of friction	35		
Slope factor	0.877		
Nominal stone diameter	0.1		
Z-function	0.078		




```
@RISK: Simulation Statistic
12-Jan-1995                  SIM
Worksheet: VELOC.WK3         (Sim #3 in Cell A:B22)
========================== =====================
Expected/Mean Result =                0.08016073
Maximum Result =                       0.1506278
Minimum Result =                     -0.04182624
Range of Possible Results =            0.1924541
Chance of Positive Result =                 99.1
Chance of Negative Result =                  0.9
Standard Deviation =                  0.02724661
Skewness =                            -0.4874924
Kurtosis =                              3.453711
Variance =                          0.0007423776
ERRs Calculated =                              0
Values Filtered =                              0
Simulations Executed =                         3
Iterations =                                1000
```

As was mentioned before, the main advantage of a program like @RISK is the fact that the same model can be used for deterministic and probabilistic calculations. Except from this advantage, there are some other advantages:

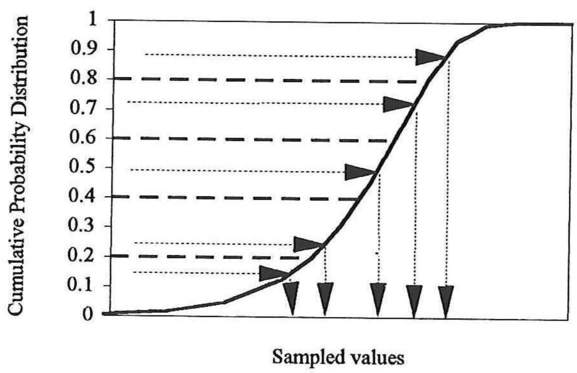
Accuracy

This method of calculation is more accurate than the level II calculations, providing that enough iterations are made. This is mainly because the real probability distributions are used instead of approximated normal distributions. If the actual Z-function has a shape that is very different from the bell-shape of a normal distribution, the difference between the level II and the level III calculation can be considerable. The large amount of iterations that have to be done to obtain this accuracy has always been the constraint for this method of calculation. With the recent developments in calculation speed of personal computers and the use of an improved method of sampling, the Latin Hypercube method, this has become a much less important constraint. Still, for a complex calculation model, a simulation run of 10.000 iterations can take 10 to 15 minutes using a computer with a 80486 / 66Mhz configuration.

BOX 2 Latin Hypercube method of sampling

Latin Hypercube sampling is a recent development in sampling technology designed to accurately recreate the input distributions through sampling in fewer iterations when compared with the Monte Carlo method. The key to Latin Hypercube sampling is stratification of the input probability distributions. Stratification divides the cumulative curve into equal intervals on the cumulative probability scale (from 0 to 1). A sample is then randomly taken from each interval or "stratification" of the input distribution. In Figure 33 an illustration is shown of a simulation run with five iterations and therefore five stratification intervals.

Figure 33 Illustration of Latin Hypercube method of sampling



This technique is called "sampling without replacement". The number of stratifications is equal to the number of iterations performed. Because the model is forced to take samples from each stratification, unwanted clustering of samples never occurs and the low probability outcomes in the tails of the distributions are better represented. The Latin Hypercube therefore has a greater sampling efficiency and faster runtimes than the Monte Carlo method of sampling for the same level of accuracy.

Stability and flexibility of the calculations

The Z-functions and the PDFs can be programmed as discontinuous functions, there is no risk for numerical problems[14] during the simulation run. The calculations have the same constraints as regular spreadsheet calculations. This gives the user the freedom to make censored or other customized PDFs with the use of standard spreadsheet functions. Some truncated distributions are provided as standard functions in @RISK. Furthermore, various parameters can be defined with a certain correlation in a correlation matrix.

Creating new probability distributions

In @RISK a large number of probability distributions are available. However, because this program has been developed also for use in industrial and financial fields of study, some specific engineering-distributions are not available. Because of the characteristics of the Monte Carlo method of sampling every probability distribution that has an exact or approximated cumulative distribution function which can be inverted, can be constructed with the uniform probability distribution (see BOX 3).

BOX 3 Defining new distributions in @RISK

The Monte Carlo sampling method works as follows:

1 A random number generator generates a number between 0 and 1 with equal probability.

2 This sample can be interpreted as a certain probability outcome of the Cumulative Distribution Function (CDF), F(x), of a certain distribution, for a certain value of variant x. With the inverted CDF this x can be calculated. For a large number of samplings, all the x-values can be combined in a histogram or Probability Mass Function which approximates the Probability Density Function (if N→∞, this should theoretically be equal).

Example Weibull-distribution

$$F(x) = 1 - e^{-(x/\beta)^{\alpha}}$$

CDF of @RISK-function @WEIBULL(α,β)

$$X_i = \beta\ \left[-\ln(1-F_i)\right]^{1/\alpha}$$

F_i is the number generated by the random number-generator which can be made with the @RISK-function: @UNIFORM (0,1). X_i is the value returned by this function

If a distributions is not provided in the program @RISK it can be constructed with a random number generator provided that its CDF is available and can be inverted.

Example Gumbel - distribution

$$F(x) = \exp\left[-\exp\left[-\frac{x-\beta}{\alpha}\right]\right]$$

CDF of not-available function: GUMBEL(α,β)

$$X_i = -\alpha \ln\left[-\ln(F_i)\right] + \beta$$

F_i is the number generated by the random number-generator which can be made with the @RISK-function: @UNIFORM (0,1)

[14] these are problems like "out of range", "run-time errors" and convergence problems

19. Acceptable probability of failure

The acceptable probability of failure of the top event in the fault tree is usually determined by regarding two different points of view: individual and societal. These two points of view can eventually be compared with observed accepted risk-levels for various activities world-wide and be used as a guide to define the acceptable probability of failure to be applied for bank protection works along the Mekong river.

19.1. Individual point of view

From the point of view of an individual, the accepted probability of failure of a bank protection can be defined as:

$$P_{f,acc} = \beta \frac{P_d}{P_{d|f}}$$

Pf,acc	=	annual acceptable probability of failure	[yr-1]
Pd	=	annual average probability of mortality	[yr-1]
Pd|f	=	probability of mortality in case of failure	[-]
b	=	policy factor (between 0.1 and 10)	[-]

In Thailand the probability of mortality due to an accident was about 5.0e-4 in 1990 of which about 1.5 10-4 was due to transport accidents15. It can be assumed that a higher probability applies to Bangkok circumstances whereas a lower value applies to less urbanized areas in Thailand, but in this case an average value of 5.0e-4 can be adopted as a conservative (on the safe side) value for areas along the Mekong river.

The probability of mortality in case of failure of a bank protection, in case an individual is in the immediate area, is believed to be about 1.0e-2.

DYING IN CASE OF FAILURE

The probability of an individual dying when present near the failing bank protection is very small. There will often be some kind of **warning** of the failure for the individual and therefore a possibility to leave the immediate area. But even in case of sudden failure there is no great danger. The **failure mode** will often be a slip circle or slide. The individual will therefore not fall from great height. It is more likely that he will tumble down the slope and possibly enter the river. The river is not wild, even in high flow season, therefore the only serious danger is that of **drowning** in case the individual can not swim.

The protected banks are most often reclaimed land without housing but with a boulevard, therefore the collapse of a house in case of failure of the protection is very unlikely. A more realistic scenario would be a car passing on the road along the protection, which would tumble down into the river with possible casualties. Therefore the event of an individual dying in case of failure is very unlikely. But how unlikely is unlikely? One in a hundred, or maybe one in a thousand? This 'guesstimate' is one of the most subjective parts of such a calculation.

[15]Thailand in Figures, Alpha Research Co.,Ltd., 1994.

The present annual average probability of mortality reflects the probability accepted by the average individual in the society at this moment. The policy factor reflects two aspects of this probability. Firstly, it reflects the possibility to deviate from the annual average probability of mortality, based on whether the activity was voluntary or not. It is common to accept smaller probabilities of failure for activities that are less voluntary. Secondly, if probabilities of failure are always evaluated against historical values, the country as a whole would, theoretically speaking, never become safer. This might not be according to the policy maker's goal to reduce or allow for a greater probability of failure. The policy factor used here expresses these policies towards failure of the bank protection and is taken as 0.1 which is the most risk-averse (=reluctant to take risk) limit.

For the MRBES this results in an annual acceptable probability of failure of:

$$P_{f,acc} = 0.1\frac{5.0e-4}{1.0e-2} = 5.0e-3$$

19.2. Societal-economical point of view

From a societal point of view there are again two ways of deriving the acceptable probability of failure. The first approach reflects the societal aversion for a large number of fatalities.

$$P_{f,acc,soc} = \frac{\beta^2 100^2 N_A}{k^2 N_{d|f}^2}$$

$P_{f,acc}$	=	annual acceptable probability of failure	[yr^{-1}]
k	=	safety threshold	[-]
N_A	=	number of places where the activity takes place (1)	[yr^{-1}]
$N_{d\|f}$	=	number of fatalities in case of failure	[-]
β	=	policy factor (between 0.1 and 10)	[-]
100	=	reference value	[-]

In this case, however, this criteria is not very important[16] and will not be discussed further. In Figure 35 the line described by this equation is shown however to give a complete picture of the safety philosophy.

The second approach is the societal-economical point of view where $P_{f,acc}$ is equivalent to the probability for which the total cost of the project will be minimal. The total cost includes the direct investment and the present value of all future costs and possible damages. The investment can be defined as a function of the probability of failure. Often a log-linear relationship is assumed:

$$I = I_o - I' \log(P_f)$$

I	=	total investment cost	[million Baht]
Io	=	fixed investment cost (mobilization cost etc.)	[million Baht]
I'	=	variable investment cost	[million Baht]
Pf	=	annual probability of failure	[yr-1]

[16] In the case of river bank protections along the Mekong river, the probability of a large number of deaths (>10) is very small. The Mekong river hardly has any flood plains, most of the land along the river is on a higher level then the flood levels: **the protections are not flood protections**

This function can be determined by varying certain design parameters such as stone diameter and fill volume, and calculating the investment cost and probability of failure for each value. For several of the already constructed protections this function could be approximated by:

$$I = 30 - 10 \log(P_f) \qquad \text{(I in million Baht)}$$

> ***COST - SAFETY FUNCTION***
> Determining the cost-safety function of a certain type of protection is one of the most difficult and most time-consuming aspects of this analysis. It demands that the cost-dimension functions are known and the dimension-safety relation can be adequately calculated for the most important parameters.

The present value of all future costs and damages of the project can be expressed as

$$C = \sum_{n=1}^{N} \left[\frac{P_f \; (S_f + N_{d|f} \;\; V_l) + M(P_f)}{(1 + r - i - g)^n} \right]$$

Pf	=	probability of failure	[yr-1]
S_f	=	expected damage in case of failure	[million Baht]
$N_{d|f}$	=	expected number of fatalities in case of failure	[-]
V_l	=	value of a human life	[million Baht]
M_{Pf}	=	maintenance cost (function of Pf)	[M Baht/yr]
r	=	interest rate, ± 10% for Thailand	[-]
i	=	inflation rate, ± 5% for Thailand	[-]
g	=	growth rate of the economy, ±7.5% Thailand, ±5% (est.) North-east	[-]
N	=	expected (technical) life of the structure, usually 30-40 years	[yr]

Several considerations will have to be taken into account

- In the design, the no-maintenance option is considered, therefore M(Pf)=0
- The expected number of fatalities in case of failure is negligible
- The factor: (r-i-g) in the formula, has a value of 0 in this case

therefore the equation reduces to

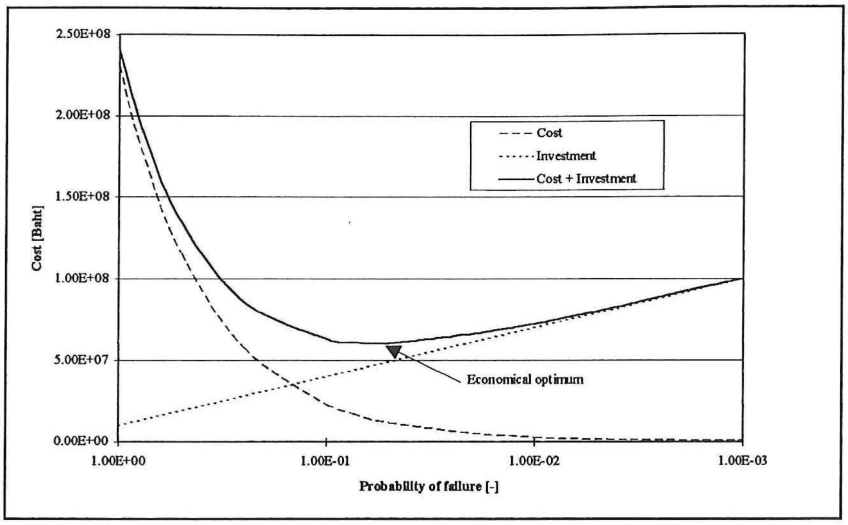
$$C = N\left[P_f S_f\right]$$

Minimizing the sum of investment and future cost with

$$\frac{d(I+C)}{d\,P_f} = 0$$

results in the optimal probability of failure for the considered scenario of I and C. In Figure 34 this optimization is shown.

Figure 34 Economically optimal probability of failure



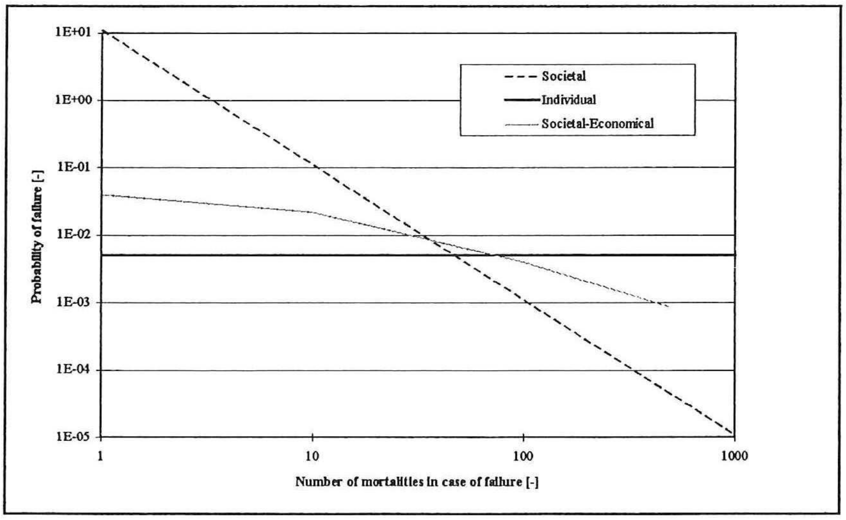
For various expected levels of damage the optimal probability of failure can be determined in the same way. The results are shown in Table 5.

Table 5 Expected probability of failure

expected damage S [million Baht]	optimal probability of failure P_f for the considered expected damage [yr-1]
250	2.9e-2
1,250	5.8e-3
2,500	2.9e-3

These approaches can be combined in a graph, see Figure 35. In this graph the area that is considered to contain safe levels of risk is bounded by the three lines. Because no large number of fatalities can be expected, only a part of this graph is of interest, the most left part. To plot the curve of economic optimal probabilities we have to convert the monetary loss to a loss in lives. In other words: the loss of how much money is equally unacceptable for society as the loss of a life? In the graph shown in section II (figure 8) the value of one life is set equal to one million dollar. If we adopt the same ratio, the equivalent amount of Baht would be 25 million. The values in Table 5 can be converted and plotted in Figure 35.

Figure 35 Acceptable probability of failure



According to the graph, an annual probability of failure of 5.0e-3 would be an acceptable risk level. One should keep in mind, however, that the analysis contains a certain number of uncertainties. *Therefore, a value of 1.0 10-3 is considered to be more appropriate to be used as an acceptable annual probability of failure for bank protections along the Mekong river.*

20. Conclusions and Recommendations

The most important aspect of the probabilistic analysis is the identification of the uncertainty of observed- and derived parameters[17]. To illustrate the techniques available and the problems encountered, parts of the MRBES were used as examples. For some of the observed parameters, large time-series were available. After statistical checking of these series, probability distributions could be derived to represent the occurence of daily and extreme values. Allthough many methods of data-checking and data-fitting have been developed since a long time already, no rigorous statistical criteria exist on which a qualitative comparison of distributions can be based and therefore the choice of a certain distribution remains, to some extent, subjective.

If data-series are surrounded with much uncertainty, like the reported sediment transport data, the (absolute) consistency with neighbouring stations should be checked before using this data in a probabilistic analysis. For the example of the sediment transport data there proved to be discrepancies, probably resulting from the fact that the data came from two different sources. For a spatial series of a certain parameter it is even more important to look at the condition of homogeneity, before using the data to derive a probability distribution. The river width, for example, proved to be much more determined by local and global geology than by morphological processes and therefore not suitable to model with a probability distribution.

In case of large amounts of data to be analyzed, information-logistical aspects can determine the level of detail to which probabilistic analysis is reasonably obtainable within the time-constraint of the project. One aspect is that a sensitivity analysis, which is in fact a trial-and-error process, can be very time-consuming if the calculation is done with a series of small programs and that some of the programs used do not allow for much flexibility in changing input data, boundary condition or model definition. Another aspect is that the editing, calculation and graphical presentation of these large amounts of data will need a good architecture of the database-calculation environment created.

It is therefore recommended that the small programs should be integrated and made more flexible. In this study a program is developed in which several programs are integrated. One of the modules of this program, named 'CROSCALC', calculates a rating curve based on a reported cross-section and allows for easy calibration of the hydraulic gradient and the hydraulic roughness by matching this curve with the reported rating curve. Another module, named 'MORPHO', uses the results of the previous module to determine various morphological parameters. The program aims to create a data-set that is internally consistent for various levels in one cross-section and for a series of cross-sections in a longitudinal overview. This program was made with the spreadsheet program Microsoft Excel v.5.0 which allows for programming with the object-oriented programming language Visual Basic. In this way the advantages of spreadsheet-functions and structured programs can be combined. It is also recommended that the possibilities are studied to analyse data in databases directly with calculation programs in order to save laborious input-output conversion.

[17] For many people the term 'parameter' should be replaced by 'variable'. In this study, however, this term is used for both. Often the term 'distribution-' or 'model-' is added to clarify what kind of parameter is meant.

A design project will often consist of several partial studies, dealt with by different engineers. One partial study will use the parameters derived in a previous study or use the same parameters used before. During the analysis of the MRBES it was found that there was no overview of which parameters were used in which partial study and that there was hardly any agreement on the notation and method of communication of the parameters' uncertainty between partial studies. This hinders a full and consistent probabilistic design-approach. The use of a 'parameter flow diagram' and 'parameter reliability labels' are therefore recommended as possible tools to alleviate these problems.

The analysis- and design process will consist of various stages in which different types of calculations will be made. In a preliminary design, a straightforward deterministic calculation will be sufficient, but for the final detailed design, a full probabilistic calculation will have to be made. Because these calculations are usually performed with different calculation programs the 'model-infrastructure' has to be reconstructed in every program used, which is time-consuming and is more susceptible for errors. In order to avoid these problems, an 'integrated design model' is recomended which can perform deterministic calculations, sensitivity analysis and full probabilistic calculations on the same model in the same program. For this design model the spreadsheet program Microsoft Excel v.5.0 was used and the so-called 'add-in' program @RISK which performs Monte Carlo (-like) simulations in spreadsheet models.

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ANNEX A

Figures Section II

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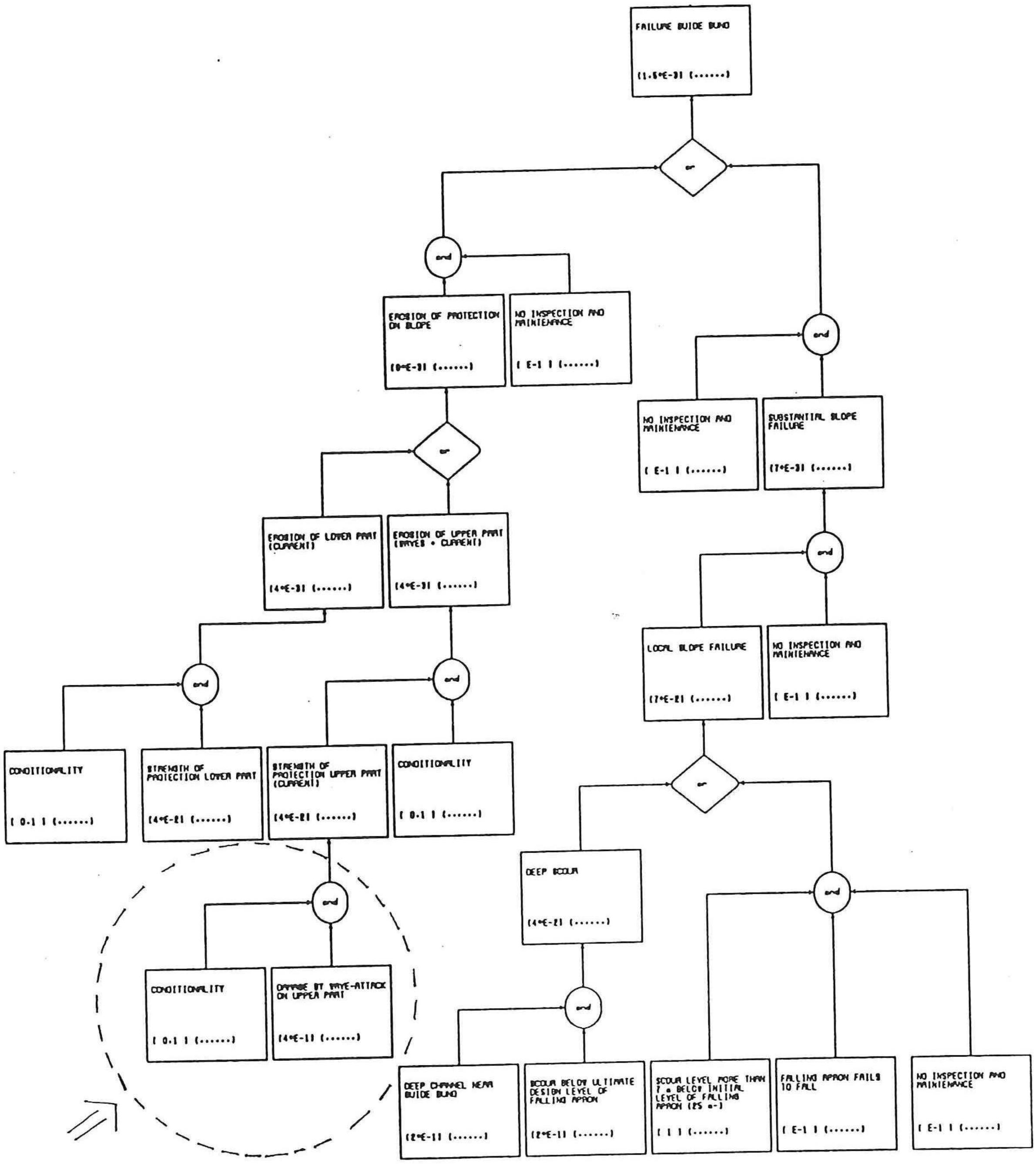
Figure 1 Fault tree guide bund Jamuna Multi purpose bridge



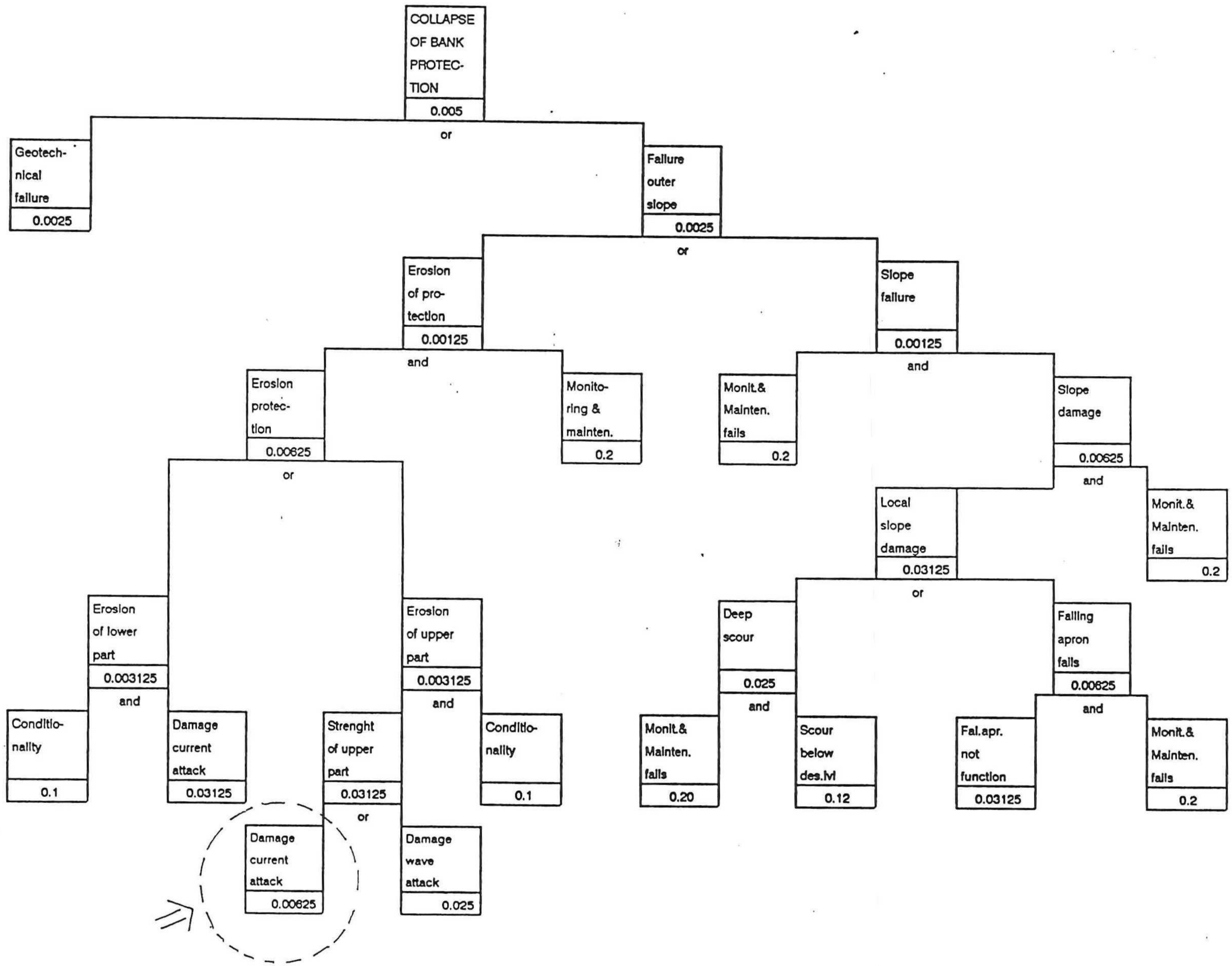


Figure 2 Fault tree river bank protection Meghna (Bhairab Bazar)

Figure 3 Influence depth factor in the design calculations of the Meghna project

HASPROB Current attack rock Chandpur Town
Beta = 2.8038
Probability of failure = 2.525304E-03

Name	Type	A	B	mu	si	x	%
Q	Gumbel	97658.000	11955.000	106827.500	16339.960	106863.200	0
i	Normal	.00	.00	2.200	.200	2.362	8
Rho_s	Normal	.00	.00	2650.000	100.000	2586.840	5
Rhowa	Normal	.00	.00	1000.000	100.000	1063.082	5
D50	Normal	.00	.00	.350	.035	.321	9
Alfa	Normal	.00	.00	15.950	1.500	16.264	1
Phi	Normal	.00	.00	40.000	4.000	39.201	1
Stbfa	Normal	.00	.00	1.000	.100	1.088	10
Kt	Normal	.00	.00	1.500	.150	1.632	10
Psicr	Normal	.00	.00	.035	.003	.032	10
Bedvl	Normal	.00	.00	24.000	2.400	25.829	7
C	Normal	.00	.00	90.000	9.000	104.790	34
Holc	Normal	.00	.00	10.000	.000	10.000	0

Z(x) = 3.311339E-04
Number of iterations = 241

HASPROB Current attack rock Chandpur Town
Beta = 4.0978
Probability of failure = 2.086679E-05

Name	Type	A	B	mu	si	x	%
Q	Gumbel	97658.00	11955.00	103103.000	14179.900	103140.700	0
i	Normal	.00	.00	2.200	.200	2.425	8
Rho_s	Normal	.00	.00	2650.000	100.000	2555.622	5
Rhowa	Normal	.00	.00	1000.000	100.000	1094.272	5
D50	Normal	.00	.00	.350	.035	.292	16
Alfa	Normal	.00	.00	15.950	1.500	16.413	1
Phi	Normal	.00	.00	40.000	4.000	38.795	1
Stbfa	Normal	.00	.00	1.000	.100	1.122	9
Kt	Normal	.00	.00	1.500	.150	1.682	9
Psicr	Normal	.00	.00	.035	.003	.031	10
Bedvl	Normal	.00	.00	24.000	2.400	26.555	7
C	Normal	.00	.00	90.000	9.000	110.099	30
Hloc	Normal	.00	.00	24.000	.000	24.000	0

Z(x) = 4.407060E-04
Number of iterations = 238

Figure 4 Probabilistic design calculations Jamuna project

CURRENT ATTACK ON DUMPED ROCK

Date :25/10/1988 Time : 9:57:25

Beta = 2.380
Prob. of failure = 8.6E-03

NAME	TYPE	A	B	C	MU	SI	X	Alfa2	dZ/dX	n
Discharge	G	60862.000	6603.000	0.000	62506.127	9763.242	69183.707	0.083	-0.000	-0.09
D_river	N	0.000	0.000	0.000	-25.000	0.010	-25.000	0.000	0.006	-0.16
rho_st	N	0.000	0.000	0.000	2600.000	25.000	2595.068	0.007	0.000	0.40
D	N	0.000	0.000	0.000	0.180	0.018	0.153	0.385	1.595	0.24
phi	N	0.000	0.000	0.000	37.000	3.000	35.878	0.025	0.002	0.09
alfa	N	0.000	0.000.	0.000	16.000	3.000	18.435	0.116	-0.005	-0.10
Psi	N	0.040	0.004	0.000	0.040	0.004	0.034	0.385	7.177	0.24

Z(X) = -0.0000

Number of iterations = 6

Calculation time = 6.75 s

Discharge	=	69183.71
Stage	=	13.82
H_river	=	38.82
Chezy	=	71.68
M_slope	=	1.19
V_current	=	3.74

Figure 5 Probabilistic reference calculations Jamuna project

```
PI=3.141593
G=9.813
KH=0.33
KT=((1+3*R)/1.3)**2
H=((Q-3220)/917)**0.5+31.04

CHEZY=15.06*(Q**0.14)
VELOC=CHEZY*(H*I*0.00001)**0.5
KSLOPE=(1.-((SIN(ANGLE*PI/180.)/SIN(PHI*PI/180.))**2))**.5
DELTA=RHOST/1000-1
DUM=(KH*KT*0.03*VELOC*VELOC)/(2*G*PSICR*KSLOPE)
Z=DN*DELTA-DUM
```

H A S P R O B Probabilistic AFDA calculations
Z-function 04

Beta = 2.5335
Probability of failure = 5.645962E-03

Name	Type	A	B	C	mu	si	x	%
Q	3	60862.000	6603.000	.000	-3180.156	48195.910	92254.700	55.11
I	1	.000	.000	.000	7.000	.001	7.000	.00
R	1	.000	.000	.000	.200	.020	.212	6.76
ANGLE	1	.000	.000	.000	16.000	1.600	16.433	1.31
PHI	1	.000	.000	.000	35.000	3.500	34.101	1.18
PSICR	1	.000	.000	.000	.040	.004	.036	15.14
RHOST	1	.000	.000	.000	2600.000	100.000	2545.903	5.26
DN	1	.000	.000	.000	.290	.029	.263	15.23

Z(x) = 4.945400E-04
Number of iterations = 101

Beta = 1.7240
Probability of failure = 4.235714E-02

Name	Type	A	B	C	mu	si	x	%
Q	3	60862.000	6603.000	.000	59591.300	15519.780	74304.480	15.93
I	1	.000	.000	.000	7.000	.001	7.000	.00
R	1	.000	.000	.000	.200	.020	.211	12.84
ANGLE	1	.000	.000	.000	16.000	1.600	16.400	2.45
PHI	1	.000	.000	.000	35.000	3.500	34.172	2.18
PSICR	1	.000	.000	.000	.040	.004	.037	28.30
RHOST	1	.000	.000	.000	2600.000	100.000	2549.689	9.91
DN	1	.000	.000	.000	.260	.026	.238	28.38

Z(x) = 1.078500E-04
Number of iterations = 101

Figure 6 Probabilistic design calculations Meghna

Current attack rip rap Bhairab Bazar
Beta = 3.5123
Probability of failure = 2.221529E-04

Name	Type	A	B	mu	si	X	%
Q	Gumbel	12971.00	1814.00	14430.140	2530.306	14474.440	1
i(*10)	Normal	.00	.00	2.000	.200	2.198	8
Rho_s	Normal	.00	.00	2600.000	100.000	2521.650	5
Rhowa	Normal	.00	.00	1000.000	100.000	1078.251	5
D50	Normal	.00	.00	.150	.015	.130	13
Alfa	Normal	.00	.00	15.950	1.600	16.527	1
Phi	Normal	.00	.00	35.000	3.500	33.781	1
Stbfa	Normal	.00	.00	1.000	.100	1.099	8
Kt	Normal	.00	.00	1.500	.150	1.649	8
Psicr	Normal	.00	.00	.035	.004	.029	18
Bedvl	Normal	.00	.00	17.000	1.700	18.282	5
C	Normal	.00	.00	70.000	7.000	82.911	28
Holc	Normal	.00	.00	25.000	.000	25.000	0

Z(x) = 8.953511E-04
Number of iterations = 78

Figure 7 Probabilistic reference calculations Meghna project

```
PI = 3.141593
G = 9.813
KT=((1+3*R)/1.3)**2

STAGE = 1. + ( Q / 1170. )**(0.67)
H = BEDLVL + STAGE
VELOC = CHEZY * (( H * ISLOPE * .00001)**(0.5))
KH = 0.33


KSLOPE=(1.-((SIN(ANGLE*PI/180.)/SIN(PHI*PI/180.))**2))**.5
DELTA = RHOST / 1000 - 1.
DUM = 0.030*KT*KH*VELOC*VELOC/(PSICR*KSLOPE*2*G)

Z = DN * DELTA - DUM
```

H A S P R O B Probabilistic AFDA calculations
Z-function 04

Beta = 3.3678
Probability of failure = 3.788819E-04

Name	Type	A	B	C	mu	si	x	%
Q	3	12971.000	1814.000	.000	12451.810	4579.642	16702.520	2.86
CHEZY	1	.000	.000	.000	70.000	3.500	74.333	14.05
ISLOPE	1	.000	.000	.000	2.000	.200	2.235	12.68
BEDLVL	1	.000	.000	.000	17.000	1.700	18.493	7.04
R	1	.000	.000	.000	.200	.020	.219	8.31
ANGLE	1	.000	.000	.000	16.000	1.600	16.737	1.94
PHI	1	.000	.000	.000	35.000	3.500	33.397	1.92
PSICR	1	.000	.000	.000	.040	.004	.034	22.11
RHOST	1	.000	.000	.000	2600.000	100.000	2513.124	6.91
DN	1	.000	.000	.000	.070	.007	.059	22.18

Z(x) = 1.400000E-05
Number of iterations = 101

H A S P R O B Probabilistic AFDA calculations
Z-function 04

Beta = 1.8634
Probability of failure = 3.120640E-02

Name	Type	A	B	C	mu	si	x	%
Q	3	12971.000	1814.000	.000	16168.300	5006.656	16476.200	3.92
CHEZY	1	.000	.000	.000	70.000	3.500	72.643	15.35
ISLOPE	1	.000	.000	.000	2.000	.200	2.146	14.34
BEDLVL	1	.000	.000	.000	17.000	1.700	17.913	7.81
R	1	.000	.000	.000	.200	.020	.212	8.91
ANGLE	1	.000	.000	.000	16.000	1.600	16.403	1.71
PHI	1	.000	.000	.000	35.000	3.500	34.167	1.53
PSICR	1	.000	.000	.000	.040	.004	.037	19.74
RHOST	1	.000	.000	.000	2600.000	100.000	2549.434	6.89
DN	1	.000	.000	.000	.050	.005	.046	19.80

Z(x) = 8.340000E-06

Figure 8 Reference calculation wave stability open stone asphalt, Jamuna study

Wave stability according to Pilarczyk (1990)

$$\Delta_m D_n = \left(\Phi_s \Psi_u \cos(\alpha)\right)^{-1} \xi_m^b H_s$$

stone density	ρ	2150 [kg/m3]	
Relative density	Δ	1.15 [-]	
accepted damage level	Sd	7 [-]	service limit state
number of waves	N	3000 [-]	
permeability factor	P	0.1 [-]	armour, filter, impermeable core
Stability factor	Φ_s	2.71 [-]	$\Phi_s = 6.2\, P^{0.18} (S^2/N)^{0.1}$
Upgrading factor	Ψ_{cr}	2 [-]	Open stone asphalt
slope (1:n)	n	3.5 [m]	
mean wave period	Tm	3 [s]	
Breaking index	ξm	1.33 [-]	plunging waves
interaction exponent	b	0.67 [-]	
Significant wave height	Hs	0.82 [m]	$\xi_m = \frac{\tan(\alpha)}{\sqrt{H_s 2\pi / gT^2}}$
Stone diameter	Dn	0.16 [m]	
Stone diameter	D50	0.19 [m]	

Wave forecasting according to Bretschneider (1977)

Fetch length	F	4000 [m]
Wind speed	u	17 [m/s]
Waterdepth	h	10 [m]
Wave height	Hs	0.82 [m]
Wave period	T	3 [s]
Wave length	L	18 [m]
Dimensionless parameters		
Wave height'	H'	0.028 [-]
Period'	T'	1.94 [-]
Fetch'	F'	136 [-]

Figure 9 Reference calculation wave stability open stone asphalt, Meghna study

Wave stability according to Pilarczyk (1990)

$$\Delta_m D_n = \left(\Phi_s \Psi_u \cos(\alpha)\right)^{-1} \xi_m^b H_s$$

stone density	ρ	2150 [kg/m3]	
Relative density	Δ	1.15 [-]	
accepted damage level	Sd	7 [-]	service limit state
number of waves	N	3000 [-]	
permeability factor	P	0.1 [-]	armour, filter, impermeable core
Stability factor	Φ_s	2.71 [-]	$\Phi_s = 6.2\, P^{0.18} (S^2/N)^{0.1}$
Upgrading factor	Ψ_{cr}	2 [-]	Open stone asphalt
slope (1:n)	n	3.5 [m]	
mean wave period	Tm	3 [s]	
Breaking index	ξm	1.27 [-]	plunging waves
interaction exponent	b	0.67 [-]	
Significant wave height	Hs	0.87 [m]	$\xi_m = \frac{\tan(\alpha)}{\sqrt{H_s 2\pi / gT^2}}$
Stone diameter	Dn	0.17 [m]	
Stone diameter	D50	0.20 [m]	

Wave forecasting according to Bretschneider (1977)

Fetch length	F	2000 [m]
Wind speed	u	23 [m/s]
Waterdepth	h	25 [m]
Wave height	Hs	0.87 [m]
Wave period	T	3 [s]
Wave length	L	17 [m]
Dimensionless parameters		
Wave height'	H'	0.016 [-]
Period'	T'	1.42 [-]
Fetch'	F'	37 [-]

Figure 10 Reference calculation wave stability open stone asphalt, Baranquilla study

Wave stability according to van der Meer (1988)

$$\Delta D = \left(6.2P^{0.18}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}\xi_m^{-0.5}\right)^{-1} H_s$$

stone density	ρ	2400	[kg/m3]	
Relative density	Δ	1.4	[-]	
accepted damage level	Sd	7	[-]	service limit state
number of waves	N	3000	[-]	
permeability factor	P	0.6	[-]	armour, no filter, no core
Stability factor	Φ_s	3.75	[-]	$\Phi_s = 6.2\, P^{0.18}\left(S^2 / N\right)^{0.1}$
slope (1:n)	n	1.5	[m]	
mean wave period	Tm	3	[s]	
Breaking index	ξm	2.88	[-]	plunging waves
interaction exponent	b	0.50	[-]	
Significant wave height	Hs	0.80	[m]	
Stone diameter	Dn	0.31	[m]	
Stone diameter	D50	0.37	[m]	

$$\xi_m = \frac{\tan(\alpha)}{\sqrt{H_s 2\pi / gT^2}}$$

ANNEX B

Figures Section III

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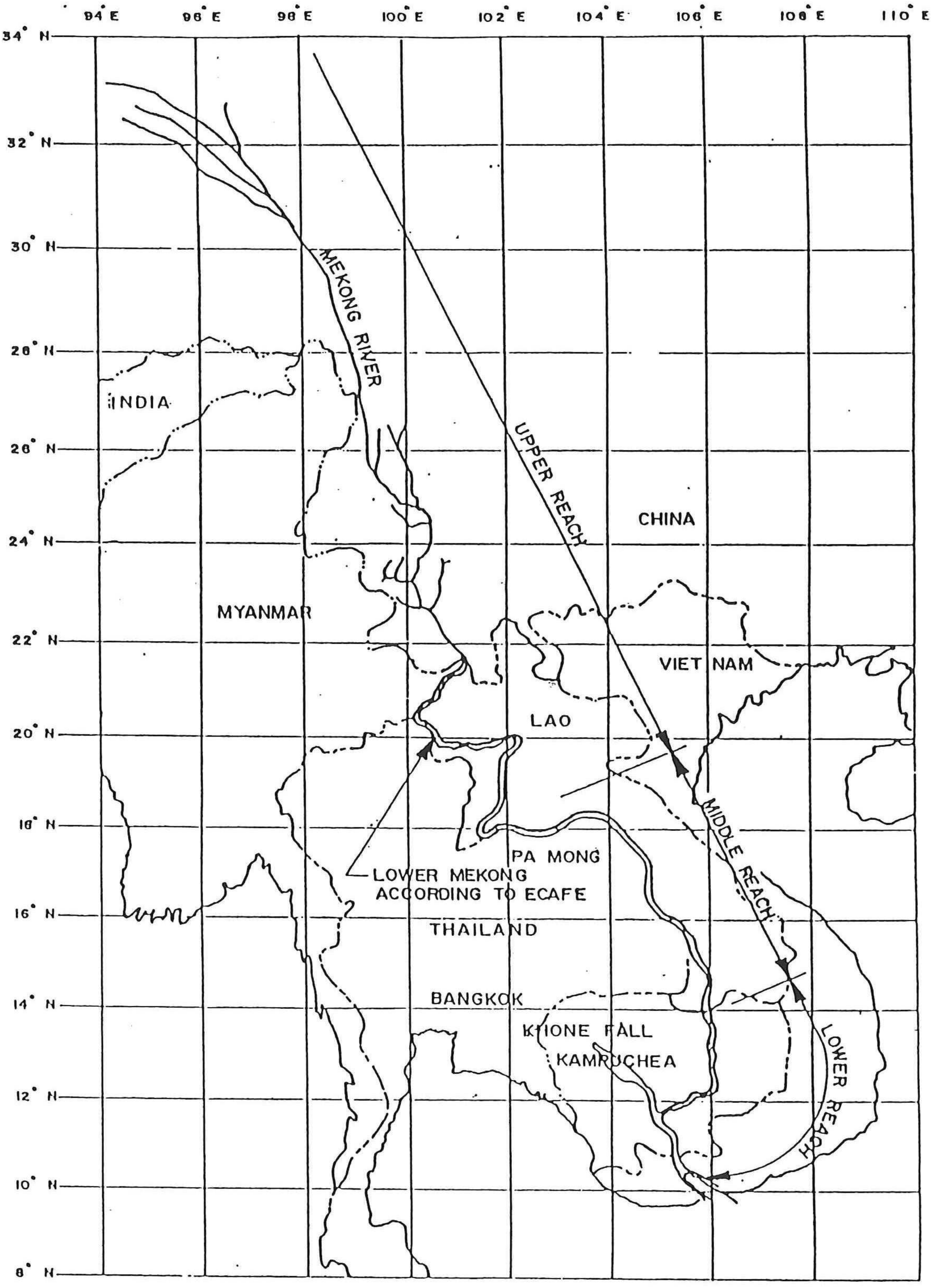
Figure 1 Upper, middle and lower reach of the Mekong river



SOURCE : DEDP.

Figure 2 Water sources and flows in the lower Mekong Basin (Mekong Committee, 1987)

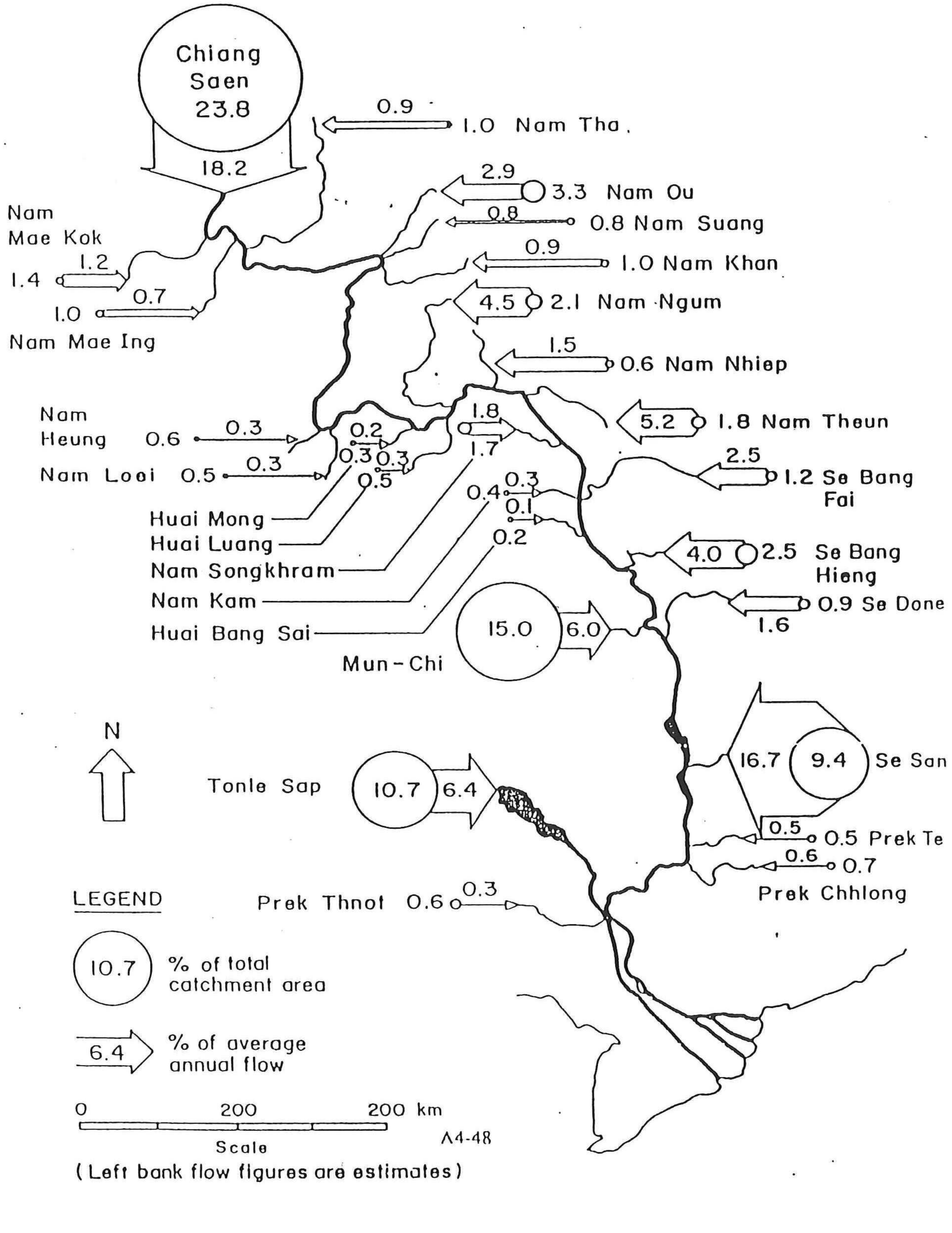


Figure 3 Chainage and gauge datum of waterlevel stations along the Mekong river

Province	Station	Chainage	Adopted gauge datum
Chiang Rai	Sop Ruak	km 2372	360.33 m MSL
	Chiang Saen	km 2364	357.31 m MSL
	Sop Kok	km 2360	355.31 m MSL
	Chiang Khong	km 2313	341.97 m MSL
Loei	Chiang Khan	km 1716	194.41 m MSL
	Ban Kok Lao	km 1700	189.50 m MSL
	Ban Pak Chom	km 1678	184.75 m MSL
	Ban Khok Wao	km 1653	172.29 m MSL
Nong Khai	Ban Sangkhom	km 1618	161.98 m MSL
	Ban Pha Tang	km 1607	160.00 m MSL
	Pa Mong Dam Site	km 1601	160.46 m MSL
	Nong Khai	km 1550	153.70 m MSL
	Phon Phisai	km 1502	150.00 m MSL
	Ban Nong Bua	km 1436	144.94 m MSL
	Ban Huai Dok Mai	km 1378	141.00 m MSL
Nakhon Phanom	Pak Huai Lang Ka	km 1300	136.00 m MSL
	Ban Chai Buri	km 1264	132.31 m MSL
	Nakhon Phanom	km 1218	130.89 m MSL
	Ban Bung Lom Tha	km 1194	130.00 m MSL
	That Phanom	km 1168	128.98 m MSL
Mukdahan	Mukdahan	km 1124	124.54 m MSL
	Ban Tha Khai	km 1107	124.00 m MSL
Ubon Ratchathani	Chanuman	km 1074	122.36 m MSL
	Khemarat	km 1040	108.03 m MSL
	Khong Chiam	km 910	89.18 m MSL
	Ban Huai Mak Tai	km 908	88.87 m MSL

BOX 1 Significance testing (Dahmen et al., 1989)

Spearman's Rank Correlation Method for testing the ***Absence of Trend***

Test statistic: $$t_t = R_{sp}\sqrt{\frac{n-2}{1-R_{sp}^2}} \quad \text{with} \quad R_{sp} = 1 - \frac{6\sum_{i=1}^{n} D_i^2}{n(n^2-1)} \quad \text{with} \quad D_i = Kx_i - Ky_i$$

n is the number of observations, Kx_i is the ranked observation and Ky_i is the original, unranked observation.

The test statistic, t_t has the Student-distribution with $v = n - 2$ degrees of freedom and is tested for the hypothesis $R_{sp} = 0$. This can be looked up in a table for the Student-distribution with the degree of freedom and required confidence level. Testing periods should not be too short (ten to fifteen years). If the time series does have a trend, the data cannot be used for frequency analysis or modeling. Removal of the trend is justified only if the physical processes underlying are fully understood.

F-Test (Fisher distribution) for testing the ***Stability of Variance***

Test statistic: $$F_t = \frac{s_1^2}{s_2^2} \quad \text{with} \quad s_m^2 = \frac{1}{n_m - 1}\sum_{i=1}^{n_m}\left(x_{m,i} - \overline{x}_m\right)^2$$ S_m is the variance of sub-set m.

The data is split into two non-overlapping sub-sets of the series. The test statistic is tested for the hypothesis $s_1^2 = s_2^2$ and can be looked up in a table for the degrees of freedom and the required confidence level. The degrees of freedom for the sub-series are $v_1 = n_1 - 1$ and $v_2 = n_2 - 1$.

t-Test for testing the ***Stability of Mean***

Test statistic: $$t_t = \frac{\overline{x}_1 - \overline{x}_2}{\sqrt{\frac{(n_1-1)s_1^2 + (n_2-1)s_2^2}{n_1 + n_2 - 2}\left(\frac{1}{n_1} + \frac{1}{n_2}\right)}}$$

The data is split into two non-overlapping sub-sets of the series. The test statistic is tested for the hypothesis $\overline{x}_1 = \overline{x}_2$ and can be looked up in a table for the degree of freedom and the required confidence level. The degree of freedom is $v = n_1 + n_2 - 2$

Double-Mass Analysis for testing the ***Relative Consistency and Homogeneity***

To determine relative consistency, the cumulative observations of a station is plotted against those from a nearby station. If there is a significant break (change of slope) in the double-mass line, the stations are not consistent. One of the great shortcomings of this analysis is that this significance is not defined. The problem of interpretation is shifted to the user.

Serial-Correlation Coefficient for testing the ***Absence of Persistence (not used in the analysis)***

Test statistic: $$r_t = \frac{\sum_{i=1}^{n-1}(x_i - \overline{x})(x_{i+1} - \overline{x})}{\sum_{i=1}^{n}(x_i - \overline{x})}$$ (= lag 1 serial-correlation coefficient)

If a series is completely random, the population's auto-correlation will be zero for all lags other than zero. The test statistic is tested for the hypothesis: $r_t = 0$. For the 5% significance level, this value should be in the range:

$$\left[\frac{-1 - 1.96\sqrt{n-2}}{n-1}, \frac{-1 + 1.96\sqrt{n-2}}{n-1}\right]$$

BOX 2 Goodness-of-fit tests used by the program "BESTFIT"

Chi-Square Test

The chi-square test for goodness of fit is a measurement of how well the observed histogram of the sample data fit an hypothized probability density function that is transformed into a probability mass function (histogram). The lower the value of the test statistic, the better the fit. The statistic that is approximately chi-square distributed is:

$$D_1 = \sum_{i=1}^{n} \frac{(y_i - F_x(x_i))^2}{F_x(x_i)}$$

where y_i is the observed value and $F_x(x_i)$ is the theoretical value. The degrees of freedom is n-r-1, where n is the number of categories of the probability mass function and r is the number of parameters estimated from the data. From the tables of the chi-square distribution, the confidence limit, $\chi^2_{\alpha, n-1}$, can be determined for any confidence level, α.

$$P\left[D_1 \geq \chi^2{}_{\alpha,k-1}\right] = \alpha$$

The choice of the intervals is very important. There are no clear guidelines for selecting these intervals; thus, in some situations, it is possible to reach different conclusions from the same data depending on the definition of these intervals.

Kolmogorov-Smirnov Test

The Kolmogorov-Smirnov test gives an indication of the goodness-of-fit of the fitted distribution to the observed values in the area of the median of the distribution. This test doesn't require the user to group the data in any way, and it is valid for any sample size n when all parameters are known. The test statistic of the Kolmogorov-Smirnov test is defined as:

$$D_2 = \max_{i=1}^{N} \{|y_i - F_x(x_i)|\}$$

where y_i are the cummulative observed values, defined as i / N_i , and $F_x(x_i)$ is the theoretical value of the fitted distribution.

$$D_2 < \frac{\alpha}{\sqrt{N}} \quad \text{for } N > 5$$

with α defined for different confidence levels.

Anderson-Darling Test

The Anderson-Darling test for goodness-of -fit is designed to detect discrepancies in the tails of distributions. It is more powerful than the Kolmogorov-Smirnov test against many different distributions. This test statistic is defined by:

$$D_3 = n \int_{-\infty}^{\infty} [y_i - F_x(x_i)]^2 \psi(x_i) f_x(x_i) dx$$

where the weight function is defined by:

$$\Psi(x) = \frac{1}{\{F(x)[1 - F(x)]\}}$$

The test statistic D_3 is the weighted average of the squared differences where the weights are largest close to either tail.

Figure 4 Source code program "Croscalc" (Visual Basic)

```
Sub croscalc()                      'A program to calculate cross-sectional
                                    'parameters for cross-sections without floodplains

Application.ScreenUpdating = False          '
Application.Calculation = xlmanual          'Improves the speed of the procedure

Set x = Range("xrange")                     '
Set y = Range("yrange")                     '
Set s = Range("slope")                      'Using the values of named ranges
Set n = Range("manning")                    'and cells in the worksheet
Set h = Range("Waterlevel")                 '

nodes = Application.CountA(x)               'No. of points of the cross-section
levels = Application.CountA(h)              'No. of levels to calculate for

'MsgBox "Make sure you define the cross-section" & Chr(13) & "without floodplains", ,
"Attention"
                                            'Message as a reminder for the user

For m = 1 To levels
    hm = h(m)

    A = 0
    P = 0
    B = 0

    For i = 1 To nodes - 1
        Select Case hm                        'Efficient way to do multiple if-statements
        Case Is >= Application.Max(y(i), y(i + 1))
            Ai = Abs(x(i) - x(i + 1)) * Abs(hm - (y(i) + y(i + 1)) / 2)
            Pi = Sqr((y(i) - y(i + 1)) ^ 2 + (x(i) - x(i + 1)) ^ 2)
            Bi = Abs(x(i) - x(i + 1))
        Case Is < Application.Min(y(i), y(i + 1))
            Ai = 0
            Pi = 0
            Bi = 0
        Case Else
            Ai = 0.5 * Abs(x(i) - x(i + 1)) *
               (hm - Application.Min(y(i), y(i + 1))) ^ 2 / Abs(y(i) - y(i + 1))
            Pi = (hm - Application.Min(y(i), y(i + 1))) / Abs(y(i) - y(i + 1)) *
               Sqr((y(i) - y(i + 1)) ^ 2 + (x(i) - x(i + 1)) ^ 2)
            Bi = (hm - Application.Min(y(i), y(i + 1))) *
               Abs(x(i) - x(i + 1)) / Abs(y(i) - y(i + 1))
        End Select
        A = A + Ai
        P = P + Pi
        B = B + Bi
    Next i

    If P = 0 Then R = 0 Else R = A / P
    If B = 0 Then havg = 0 Else havg = A / B

    Range("Width").Cells(m) = B                '
    Range("Area").Cells(m) = A                 '
    Range("Radius").Cells(m).Value = R         'Filling named ranges in the
    Range("Depth").Cells(m).Value = havg       'sheet with output values

Next m

Application.ScreenUpdating = True              'Resetting automatic screenupdating
Worksheets("Croscalc").Calculate               'and recalculation

End Sub
```

Figure 5 Velocity measurements

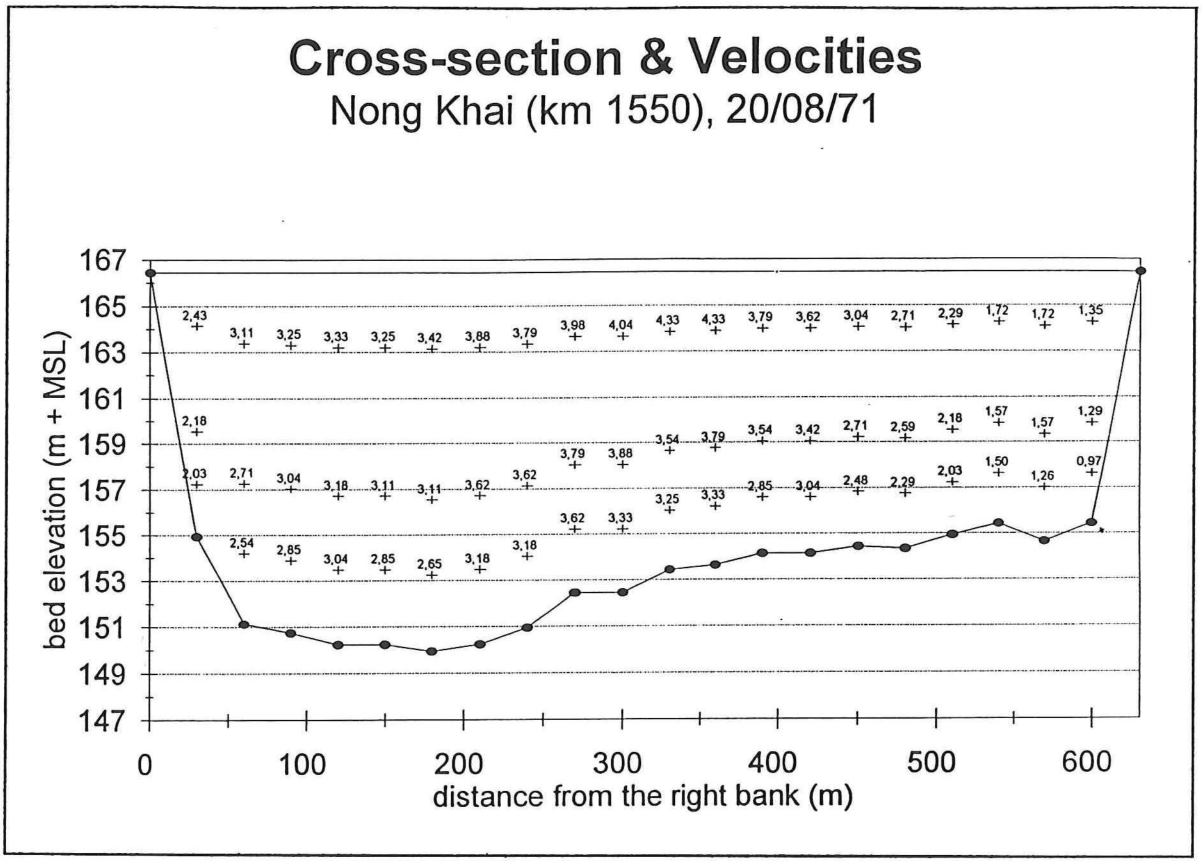
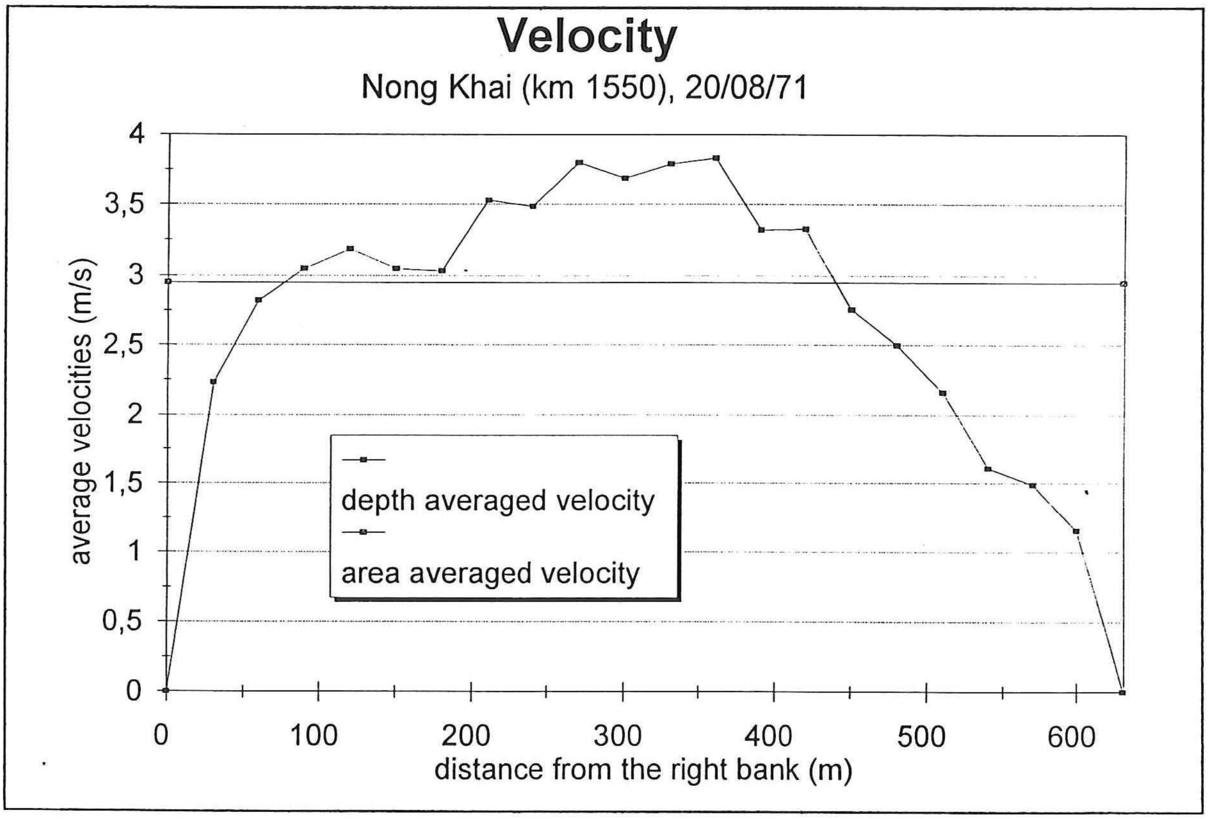


Figure 6 Area-averaged and depth-averaged velocity



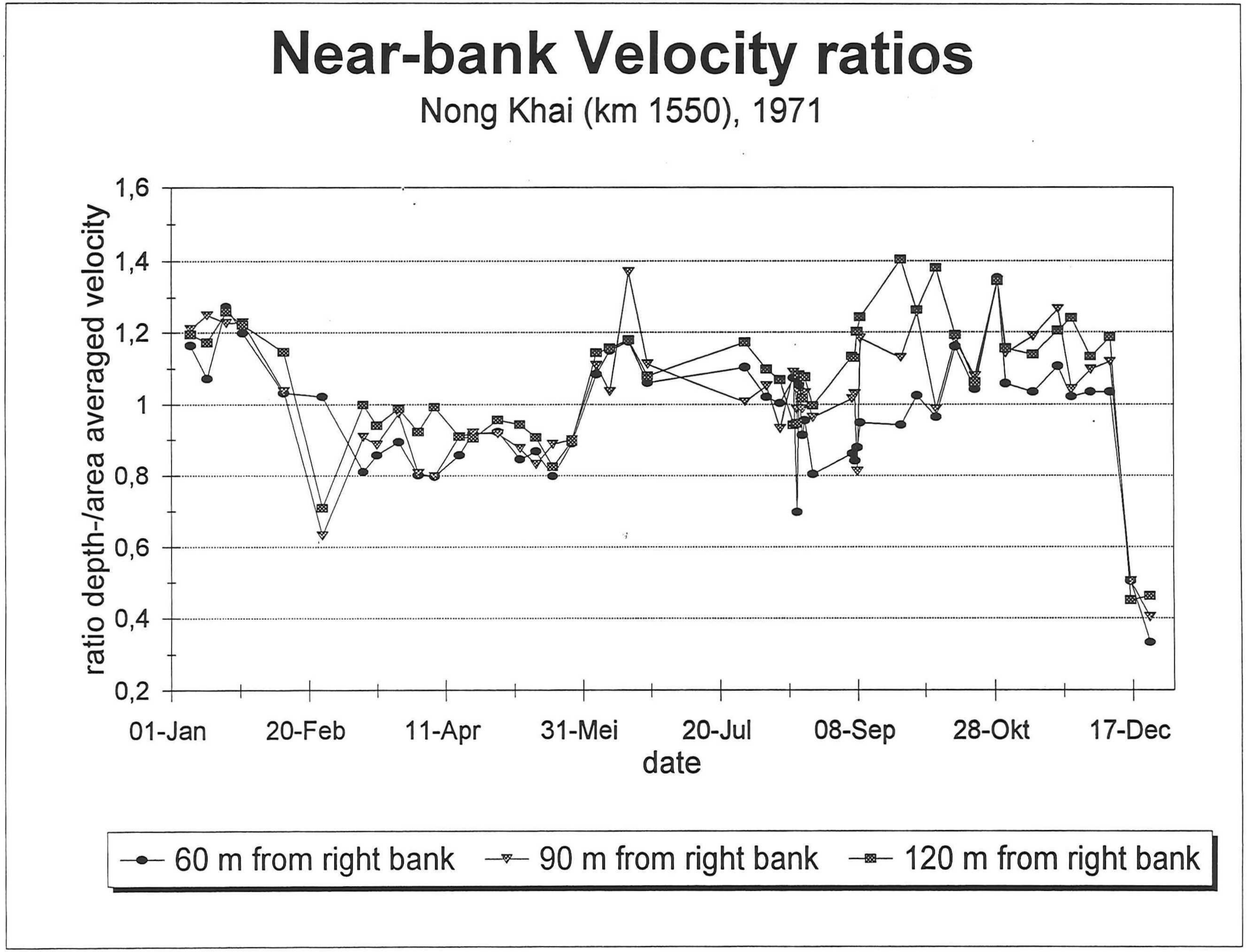


Figure 7 Near-bank velocity ratios

Figure 8 Source code program "Morpho" (Visual Basic)

```
Sub morphology()

Application.ScreenUpdating = False      'improves the speed of the procedure
Application.Calculation = xlmanual

Set s = Range("slope")
Set n = Range("manning")
Set h = Range("Waterlevel")
Set Q = Range("Discharge")
Set R = Range("Radius")
Set B = Range("Width")
Set rhos = Range("Stone_density")
Set d50 = Range("Grading_D50")
Set sed = Range("Exponent")

levels = Application.CountA(h)           'no. of levels to calculate

delta = rhos / 1000 - 1

For m = 1 To levels
    C = R(m) ^ (1 / 6) / n
    shi = R(m) * s / (delta * d50)
    If R(m) = 0 Then
        sus = 0
        lambdas = 0
        lambdaw = 0
        inter = 0
        lp = 0
        ld = 0
    Else
        If d50 > 0.001 Then
            sus = 1 / 1.1 * shi
        Else
            sus = Sqr(9.81 * R(m) * s) / (0.00001 / d50 * Sqr(1 + _
            10000000000# * delta * 9.81 * d50 ^ 3))
        End If

        lambdaw = C ^ 2 * R(m) / (2 * 9.81)
        lambdas = B(m) ^ 2 * 0.85 * Sqr(shi) / ((Application.Pi()) ^ 2 * R(m))
        inter = lambdaw / lambdas
        test = (sed + 1) * (inter - inter ^ 2 - ((sed - 3) / 2) ^ 2)

        If test < 0.1 Then
            lp = "out of range"
        Else
            lp = 4 * Application.Pi() * lambdaw / Sqr(test)
        End If

        ld = 2 * lambdaw / (inter - (sed - 3) / 2)

    End If

    Range("Shields").Cells(m) = shi
    Range("Suspension").Cells(m) = sus
    Range("Interaction").Cells(m).Value = inter
    Range("Wave_number").Cells(m).Value = lp
    Range("Damping_length").Cells(m).Value = ld

Next m

Application.ScreenUpdating = True
Worksheets("crosscalc").Calculate

End Sub
```

BOX 3 Sample statistics

Sample statistics are generally used as estimators for the population parameters. Estimators, whose expected values are not equal to the population parameters are said to be 'biased' estimators. For the first four 'sample moments', as they are called, the unbiased estimators are also given, indicated with an asterix (*). For large values of n the biased and unbiased estimators become equal. One should beware that in practical cases and in some computer programs, biased estimators are used (example: 'BESTFIT'). Spreadsheet programs like Excel and Quattro Pro calculate unbiased estimators with their standard functions.

Estimator for the mean, μ

$$\overline{x} = \frac{1}{n}\sum_{i=1}^{n} x_i$$

Estimator for the variance, σ^2

$$s^2 = \frac{1}{n}\sum_{i=1}^{n}(x_i - \overline{x})^2 \quad \text{(biased)} \qquad s^{*2} = \frac{1}{n-1}\sum_{i=1}^{n}(x_i - \overline{x})^2 \quad \text{(unbiased)}$$

Estimator for the coefficient skewness (degree of asymmetry)

$$C_s = \frac{1}{n}\sum_{i=1}^{n}\left(\frac{x_i - \overline{x}}{s}\right)^3 \text{(biased)}, \qquad C_s^* = \frac{n}{(n-1)(n-2)}\sum_{i=1}^{n}\left(\frac{x_i - \overline{x}}{s}\right)^3 \text{(unbiased)}$$

Estimator for the coefficient of kurtosis ('peakedness' or 'flatness')

$$C_k = \frac{1}{n}\sum_{i=1}^{n}\left(\frac{x_i - \overline{x}}{s}\right)^4 \text{(biased)}, \qquad C_k^* = \frac{n(n+1)}{(n-1)(n-2)(n-3)}\sum_{i=1}^{n}\left(\frac{x_i - \overline{x}}{s}\right)^4 \text{(unbiased)}$$

sometimes, the kurtosis 'relative to the normal distribution' is used :

$$C_k = \left\{\frac{n(n+1)}{(n-1)(n-2)(n-3)}\sum_{i=1}^{n}\left(\frac{x_i - \overline{x}}{s}\right)^4\right\} - \frac{3(n-1)^2}{(n-2)(n-3)}$$

BOX 4 Confidence intervals

If mean and variance are unknown, the sample statistics can be used to define a confidence interval on the mean.

The test statistic is:

$$t_t = \frac{\overline{x} - m}{s^* \sqrt{1/n}}$$ with the Student distribution

The confidence interval is

$$P\left[\overline{x} - \frac{s^*}{\sqrt{n}} t_{\alpha/2,n-1} < \mu < \overline{x} + \frac{s^*}{\sqrt{n}} t_{\alpha/2,n-1}\right] = 1 - \alpha$$

α = confidence limit
n-1=degrees of freedom
with $t_{\alpha/2,\, n-1}$ from a table of the Student distribution

If the variance is known, this reduces to

$$P\left[\overline{x} - \frac{\sigma}{\sqrt{n}} k_{\alpha/2} < \mu < \overline{x} + \frac{\sigma}{\sqrt{n}} k_{\alpha/2}\right] = 1 - \alpha$$

with $k_{\alpha/2}$ from a table of the normal distribution

Confidence intervals on the variance of a random variable can be formed once the distribution of s^2 is known. In the case of a normal distribution of variable x:

$$P\left[\sigma^2 \le \frac{ns^{*2}}{\chi^2_{1-\alpha,n-1}}\right] = 1 - \alpha$$

with $\chi^2_{1-\alpha,n-1}$ from a table of the χ^2 - distribution

BOX 5 Confidence statements encountered in literature used in MRBES

"Finally the accuracy of the measured transport rates is discussed. A comparison of flume experiments performed under similar flow conditions (equal depth, velocity, particle size, temperature) by different research workers shows deviations of the transport rates up to a factor 2. Thus even under controlled flume conditions, the accuracy of the measured values is rather low, which may be caused by the influence of the applied width depth ratio, the applied adjustment period to establish uniform flow conditions and the applied experimental method (sand feed or recirculating flume). Based on this, it is stated that it is hardly possible to predict the transport rate with an inaccuracy less than factor 2." [van Rijn, p 7.38]

"Taking a time-averaging period of 2 to 3 minutes, the relative standard deviation of the local flow velocities will be about sigma/mu=0.1. The relative standard deviation of the sediment concentration will be relatively large, especially close to the bed, say sigma/mu=0.3." [van Rijn, p 13.8]

"Most geologists can estimate the density of rock to within 100 kg/m3 from a hand specimen. In general, dealing with one type of rock in a quarry, the 90% exceedence value is not more than 100 kg/m3 less than the average density." [CIRIA/CUR, p 78]

"Gradings of rock fulfilling the class limit specification described in the following section may be expected to have standard deviations in Dn50 varying from 1% for heavy gradings to 7% for wide gradings." [CIRIA/CUR, p 94]

"The reliability of van der Meer's formula can be expressed by giving the coefficients 6.2 and 1.0 in equations (5.44) and (5.45) a normal distribution with a certain standard deviation. The coefficient 6.2 can be described by a standard deviation of 0.8 (variation coefficient 6.5%) and the coefficient 1.0 by a standard deviation of 0.08 (8%). These values are significantly lower than that for the Hudson formula at 18% for Kd^1/3 (with mean Kd of 4.5). With these standard deviations it is simple to include 90% or other confidence bands." [CIRIA/CUR, p266]

"It should also be noted that the US Geological Survey maintains a program which trains engineers in the estimation of channel resistance coefficients. The results of this program indicate that trained engineers can estimate resistance coefficients with an accuracy of 15% under most conditions (Barnes,1967)." [French, p131]