Introduction to bed, bank and shore protection

Cover:
“De machtige grijper, overwinnaar in den strijd tegen de zee” (The mighty grab, victor of the battle against the sea).
Johan Hendrik van Mastenbroek (1875 - 1945), Oil on canvas, Zuiderzeemuseum, Enkhuizen, The Netherlands.

Introduction to bed, bank and shore protection

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A little learning is a dangerous thing;
Drink deep or taste not the Pierian spring.
Alexander Pope (1688-1744)

Preface

Every book is unique. 'This one is because of a combination of two things:

- the coverage of subjects from hydraulic, river and coastal engineering, normally treated in separate books
- the link between theoretical fluid mechanics and practical hydraulic engineering.

On the one side, many fine textbooks on fluid motion, wave hydrodynamics etc. are available, while on the other side one can find lots of manuals on hydraulic engineering topics. The link between theory and practice is seldom covered, making the use of manuals without understanding the backgrounds a "dangerous thing". Using a cookbook without having learned to cook is no guarantee for a tasty meal and distilling whisky without a thorough training is plainly dangerous. Manuals are often based on experience, either in co astal or river engineering, or they are focussing on hydraulic structures, like weirs and sluices. In this way, the overlap and analogy between the various subjects is missed, which is a pity, especially in nonstandard cases where insight into the processes is a must. This book tries to bridge the gap between theoretical hydrodynamics and designing protections. Imagination of what happens at an interface between soil and water is one of the keywords. However, this can only partly be derived from a textbook. Using one's eyes every time one is on a river bank, a bridge or a beach is also part of this process. In the same sense, a computer program never can replace experimental research completely and every student who wants to become a hydraulic engineer should spend some time doing experiments whenever there is a possibility. Anyway, the purpose of this book is to offer some know how, but even more important, some know why.

The painting on the cover represents three major elements in protection against water. The inset right under pictures the power of water, symbolised by Neptune who is enthousiastically trying to enter the gate while money and knowledge, symbolised by Mercury and Minerva, respectively, are the means to stop this. The painting itself depicts the granting of the right to establish an administrative body by the people of Rhineland, a polder area, by the count of Holland in 1255. People's participation is always a major issue in hydraulic engineering, as most projects serve a public goal. People's participation and money is not what this book offers, but I do hope that it will contribute to the knowledge to be able to make durable and sustainable protections.

Gerrit Jan Schiereck, Dordrecht, December 2000

Preface to the 2nd edition

The main reason to make a 2nd edition of this book was that we run out of copies. The basic setup of the book has not been changed. Also the fundamentals did not change in the last decade. Some new findings on turbulence have been added; the chapters on execution have been updated to the latest level of technology. Also a number of new examples from the last decade have been included. Finally the book is again in line with the latest standards. To indicate that this is a new version of the book a new cover has been designed. On the first cover an allegoric painting from the office of the waterboard of Rhineland was shown. For this edition I have selected a painting of Mastenbroek (1932) depicting the closure of the Afsluitdijk. A situation where the stability of the bed material was essential for the completion of the works. The painting expresses the strength of the grab, needed to combat the strength of the water.

Henk Jan Verhagen, Delft, July 2012

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1 INTRODUCTION

Coastal protection along the Javanese coastline (photo Verhagen)

1.1 How to look at protections

1.1.1 Why and when

The interface of land and water has always played an important role in human activities; settlements are often located at coasts, river banks or deltas. When the interface consists of rock, erosion is usually negligible, but finer material can make protection necessary. In a natural situation, the interface moves freely with erosion and sedimentation. Nothing is actually wrong with erosion, unless certain interests are threatened. Erosion is somewhat like weed: as long as it does not harm any crop or other vegetation, no action is needed or even wanted. There should always be a balance between the effort to protect against erosion and the damage that would occur otherwise.

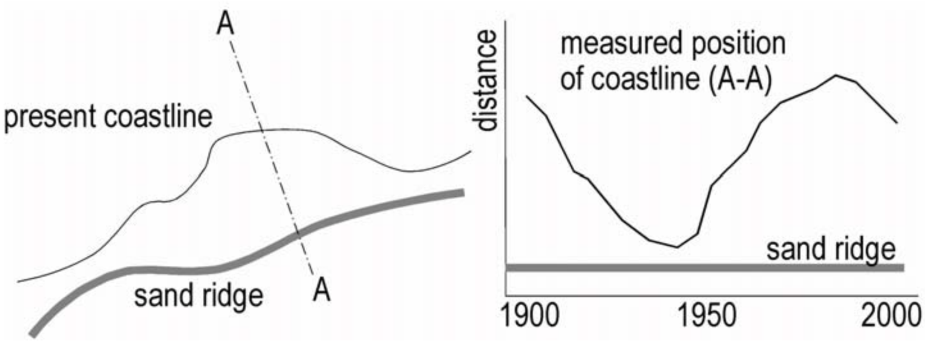

Figure 1-1 To protect or not to protect, that's the question

Figure 1-1 shows cyclic sedimentation and erosion of silt (with a period of many decades) seaward of a natural sand ridge. In a period of accretion people have started to use the new land for agricultural purposes. When erosion starts again, the question is whether the land should be protected and at what cost. Sea-defences are usually very costly and if the economic activities are only marginal, it can be wise to abandon the new land and consider the sand ridge as the basic coastline. If a complete city has emerged in the meantime, the decision will probably be otherwise. With an ever increasing population, the pressure on areas like these also increases. Still, it is good practice along a natural coast or bank to build only behind some set-back line. This set-back line should be related to the coastal or fluvial processes and the expected lifetime of the buildings. For example, a hotel has a lifetime of, say 50 years. It should then be built at a location where erosion will not threaten the building within 50 years, see Figure 1-2. So, in fact the unit for a set-back line is not meters but years! These matters are Coastal Zone Management issues and are beyond the scope of this book.

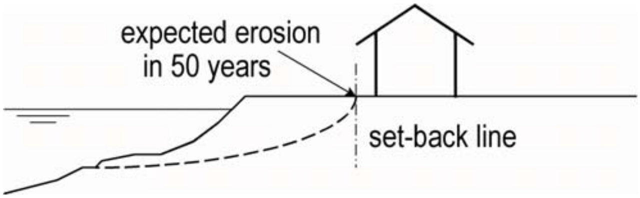

Figure 1-2 Building code in eroding area

Besides erosion as a natural phenomenon, nature can also offer protection. Coral reefs are excellent wave reductors. Vegetation often serves as protection: reed along river banks and mangrove trees along coasts and deltas reduce current velocities and waves and keep the sediment in place. Removal of these natural protections usually mark the beginning of a lot of erosion trouble and should therefore be avoided if possible. So, a first measure to fight erosion, should be the conservation of vegetation at the interface. Moreover, vegetation plays an important role in the ecosystems of banks. Chapter 12 deals with these aspects and with the possibilities of nature-friendly protections.

Finally, it should be kept in mind that, once a location is protected along a coast or riverbank that has eroded on a large scale, the protected part can induce extra erosion and in the end the whole coast or bank will have to be protected. So, look before you leap, should be the motto.

A lot of cases remain where protection is useful. Figure 1-3 gives some examples of bed, bank and shore protections. Along canals, rivers and estuaries, bank protection is often needed to withstand the loads caused by flow, waves or ships. Shore protection structures include seawalls, revetments, dikes and groynes. Bed protection is necessary where bottom erosion could endanger structures, like bridge piers, abutments, in- or outlet sluices or any other structures that let water pass through.

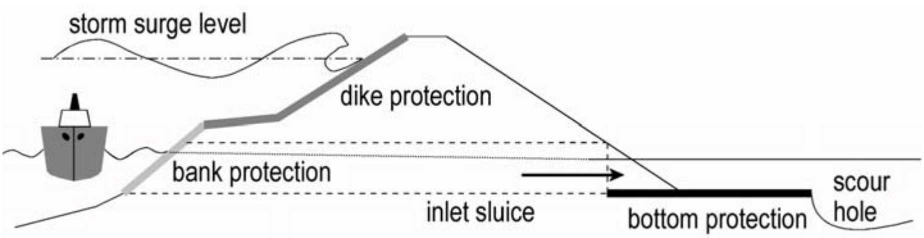

Figure 1-3 Examples of protection

1.1.2 Design

Protections of the interface of land or soil and water are mostly part of a larger project: e.g. a navigation channel, a sea defence system, an artificial island or a bridge. Therefore, the design of a protection should be tuned to the project as a whole, as part of an integrated design process. In general it can be said that the resulting design should be *effective* and *efficient*. Effective means that the structure should be functional both for the user and the environment. This implies that the structure does what it is expected to do and is no threat for its environment. Efficient means that the costs of the (effective) structure should be as low as possible and that the construction period should not be longer than necessary.

A design that combines effectiveness and efficiency can be said to be "*value for money*" The intended value becomes manifest in the *terms of reference* (ToR) which contains the demands for a structure. This ToR has to be translated into concepts (possible solutions). Demands and concepts do not match one to one and a fit between the two is to be reached with trial and error. Promising concepts are engineered and compared. One comparison factor, of course, is costs. The designer's task to get value for money can be accomplished by compromising between four elements, see Figure 1-4.

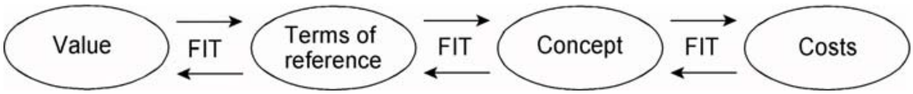

Figure 1-4 Value for money

The design process is of a cyclic nature because it is impossible to go directly from left to right in Figure 1-4. In the first phase, the designer works with a very general notion of the ToR and with some concepts in mind, based on his own or others' experiences. An integrated design process starts with a rough approach to all four elements in Figure 1-4, refining them in subsequent design phases. Effectivity can be evaluated in terms of functionality, environment and technology, while efficiency is expressed in terms of costs and construction although, of course, there are several overlaps and links between these aspects. They all play a role in each of the design phases, but the focus gradually shifts as indicated in Figure 1-5.

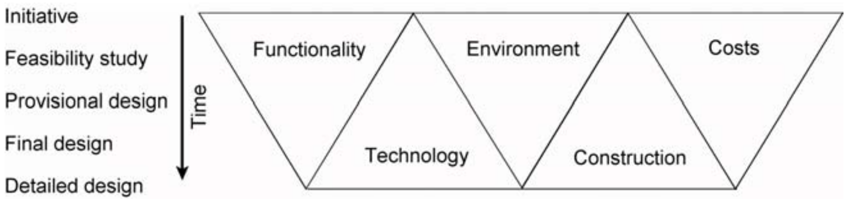

Figure 1-5 Focus during design process

Level of detail

In any project it is possible to discern various levels of detail. It is good to be aware of the level of detail one is working on and to keep an eye on the adjacent levels. An example of these levels (other divisions are, of course, possible):

1. System (Macro level)
2. Components (Meso level)
3. Parts (Mini level)
4. Elements (Micro level)

Examples of the macro level are e.g. a coastal zone, a water system (river, lake etc.) a harbour or a polder. On the meso level, one can think of components like a sea defence (dike, sea wall etc.), a river bank, a breakwater, a closure dam or an outlet sluice. On the mini level we look at dike protections, bank protections or bed protections. The micro level, finally, consists of elements like stones, blocks etc. In this hierarchy, the title of this book indicates that it treats subjects on the third level. Level 1 should always play a role in the background, see *e.g.* Section 1.1.1. Level 2 will be treated where and when adequate, while sometimes level 4 also plays a role e.g. when it comes to defining stone sizes. As a consequence of these levels, it can be said that the design of protections in a large project is usually more in the lower part of Figure 1-5, when it comes to the technical development of a plan.

1.1.3 Science or craftsmanship

Protections of the interface of land and water have been made for more than 1000 years. Science came to this field much later, as a matter of fact very recently. The second world war boosted the understanding of waves and coasts. In the Netherlands after 1953, the Delta project had an impact on the research into protection works. In the last decades, major contributions to the design practice have been made, thanks to new research facilities, like (large scale) wind wave flumes, (turbulent) flow measurement devices, numerical models etc. progress has been made in The scientific basis of our knowledge has progressed considerably, but even after 50 years, much of the knowledge of these matters is of an empirical nature. Most formulas in this book are also empirical, based on experiments or experience.

Working with these empirical relations requires insight, in order to prevent misconceived use. The idea underlying this book is to start with a theoretical approach of the phenomena, focussing on understanding them. In the design of protections, especially in the unusual cases, a mix of science and experience is required. Since undergraduates, by definition, lack the latter, a sound theoretical basis and insight into the phenomena is paramount. This book goes one step further than simply presenting empirical design relations; it aims to create a better understanding of these relations. Engineering is an applied science, which then, by definition, means that science is the basis but not the core. Creativity, experience and common sense are just as important.

Computer models play an increasing role in engineering. For a hydraulic engineer, however, a sheet of white paper and a pencil are still essential, especially in the preliminary stage of a design. A hand made sketch of a current or wave pattern is as valuable as the correct application of calculation rules. For both, a good insight into the physics of the processes involved is indispensable.

1.2 How to deal with protections

1.2.1 Protection against what?

Interfaces between land and water exist in all sizes and circumstances. Figure 1-6 gives an idea of typical values for the loading phenomena in various water systems (of course it is always possible to find an example with different figures).

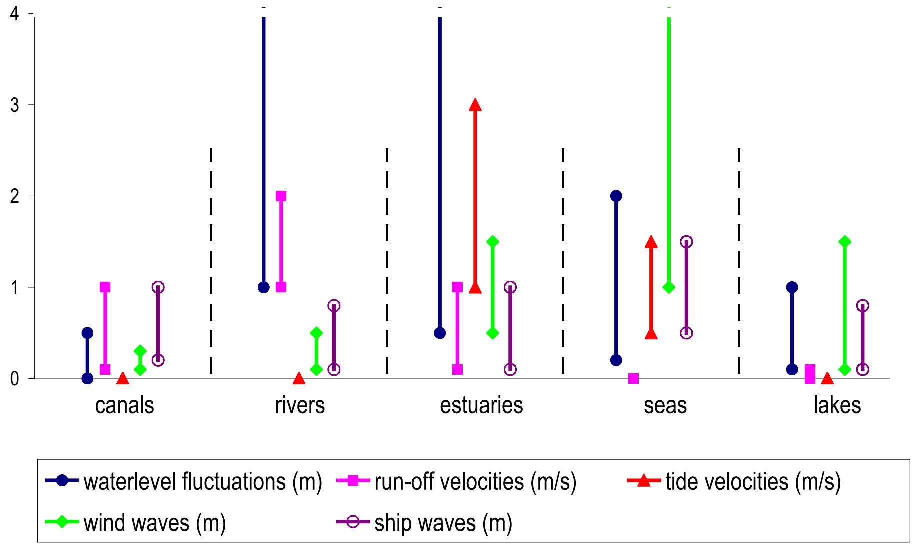

Figure 1-6 Hydraulic conditions in water systems

This book treats the interface stability by looking at the phenomena instead of the water systems. This is more exceptional than it seems, because most textbooks deal with shore protection or river training works or navigation canals etc. Much of the knowledge of these protection works is based on experience and experience is often gained in one of the mentioned fields, not in all of them. This is a pity because many of the phenomena involved are similar: ship waves and wind waves have different sources, but behave very much the same. The same holds for flow in a river, through a tidal closure or an outlet sluice, when it comes to protecting the bed or the bank. Moreover, in river bank protections, wind waves can sometimes play a role, which is often neglected in textbooks on river engineering. Therefore, an attempt is made to find the physical core of all these related problems.

One thing protections have in common, is that their function is to ***withstand the energy of moving water***. Water in motion contains energy: currents, wind waves, ship movement, groundwater-flow etc, which can become available to transport material. The energy comes from external sources, like wind or ships, and eventually ends up as heat by means of viscous friction. This is not an energy loss but an energy transfer, from kinetic energy, via turbulence, to heat. Turbulence plays an important role and will be discussed in more detail in the next chapter. For now it is sufficient to say that turbulence is related to the transformation of kinetic energy into heat. During this transfer, turbulence contributes to the attack of the interface.

Hydraulic engineering research is often empirical and fragmented. This leads to an avalanche of relations for each subject, while the connections remain unclear. One of the basic ideas of this book is to show similarities and differences between the various phenomena and therefore between the various formulas, in order to clarify the overall picture. Chapter 2 deals with open-channel flow, Chapter 5 with porous flow (flow through pores of granular structures like soil or rock), Chapter 7 with waves and Chapter 9 with ships. These subjects can and will be treated separately, but there are more similarities than many textbooks reveal.

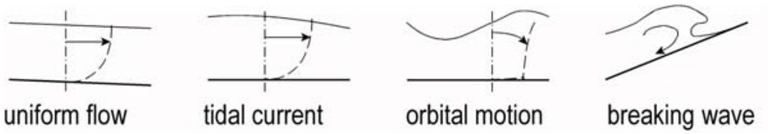

Figure 1-7 Flow and wave situations

Uniform flow is the starting point for many hydraulic considerations, see Figure 1-7. The equilibrium between gravity and wall friction completely determines the flow. The boundary layer, connected with the wall friction, takes up the whole waterdepth, is turbulent and shows a logarithmic velocity profile. The velocity profile of tide waves (very long waves with typical periods of 12 hours and wave lengths of several hundreds of km's) only slightly differs from the uniform flow velocity profile. It is therefore justified, when designing a protection, to consider tidal currents as a succession of uniform flow situations with different velocities. For wind waves (typical periods of 5 - 10 s and wave lengths of 50 - 150 m), the situation is completely different with a non turbulent orbital motion and a thin turbulent boundary layer, although such a wave in very shallow water will again approach the situation with a tidal wave. Finally, a wave that breaks on a slope, leads to turbulence over the whole waterdepth.

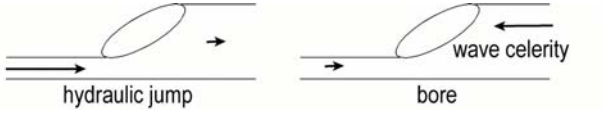

Figure 1-8 Jump and bore

A hydraulic jump and the roller of a broken wave (the bore) are very much the same. This can be seen when the jump is observed from a fixed position and the bore from a position that moves at wave celerity. The turbulence characteristics, caused by the friction between roller and flowing water, are also similar. Chapter 7 will show this in detail.

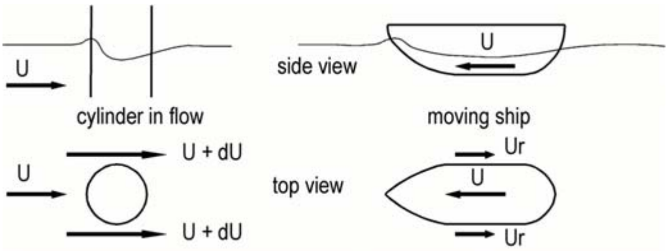

Figure 1-9 Flowing water versus moving object

The same similarity exists between a fixed object in flowing water and a ship sailing in still water. The water around the object accelerates, while around the ship a return current occurs, both leading to a water level depression.

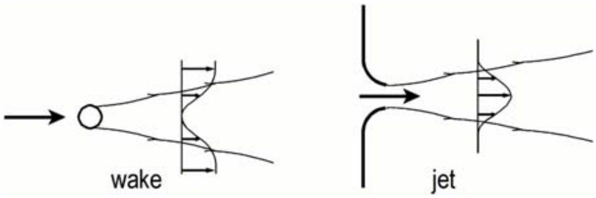

Figure 1-10 Mixing layers in wake and jet

Behind an object in flow or behind a ship, a wake occurs, where there is a velocity deficit compared with the environment. This velocity difference causes a so-called mixing layer where relatively slowly and quickly moving water mix which leads to a lot of turbulence. In a jet (an outflow in stagnant or slowly moving water) the same velocity differences (but now due to an excess velocity) occur, causing the same mixing layer and turbulence.

The last analogy in this chapter is between pipe flow and porous flow (see Figure 1-11) which is flow through a porous medium like sand or stones. In a straight pipe the (uniform) flow is determined by the wall friction. In an irregular pipe, uniform flow will never really occur, due to the irregularities in the cross-section. Even with a constant discharge, accelerations and decelerations will always occur and, at sharp discontinuities, even flow separation with a mixing layer will take place. The flow between grains, when considered on a micro level, also show continuous accelerations and decelerations and the same basic equations describe this type of flow, including laminar and turbulent flow. In practice, however, the flow is integrated over many grains and pores, because it is not feasible and not necessary to have velocity information of every pore.

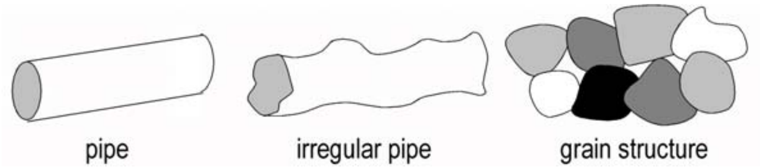

Figure 1-11 Pipe flow and porous flow

All of the above examples contain elements of three phenomena: *wall flow, mixing layer and oscillating flow (wave)* with turbulence playing a role in all three of them. Wall flow is present in uniform (pipe) flow, tidal flow and in the boundary layer of wind waves. A mixing layer is visible in hydraulic jumps and bores (between main flow and roller) and in wakes and jets. On a micro scale, porous flow includes both wall flow and mixing layers. It is a simplification to say that every flow situation can be reduced to these three basic phenomena or a combination of them but every situation contains at least one of these three features. It is therefore indispensable to be able to recognize and understand their elementary properties. This is what the chapters on loads are about: Chapter 2 on flow, Chapter 5 on porous flow and Chapter 7 on waves.

Hydraulics and geotechnics

In general, hydraulic and soil-mechanical mechanisms determine the stability of a structure. Cause and effect can lie in both fields: failure of a protection can cause settlements of a structure, but vice versa is also possible. Figure 1-12 gives some examples.

In case (a) the sill under a water-retaining structure is a malfunctioning filter. Due to erosion, the structure will settle. As the maximum gradient inside the filter possibly occurs at the entrance side of the flow, the settlement can be against the head difference. In case (b) a canal is situated above groundwater-level. To prevent water losses, the bottom of the canal is coated with an impermeable protection. If the dike along the canal settles, due to insufficient strength of the subsoil, a fracture in the protection can occur and the canal water can drain into the subsoil. Case (a) looks like a soil mechanical problem but it has a hydraulic cause while in case (b) it is the other way around.

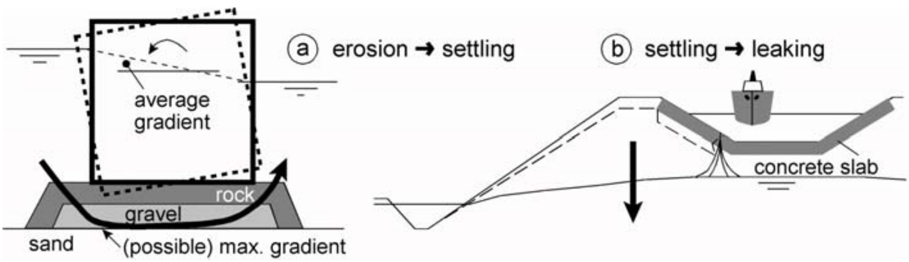

Figure 1-12 Cause and effect

1.2.2 Failure and design

The previous section already stressed that insight in phenomena is paramount for the design of a reliable interface protection. Neglect of a relevant phenomenon can lead to a protection that causes more damage than it prevents or that shifts the problem to the neglected phenomenon, see Figure 1-13.

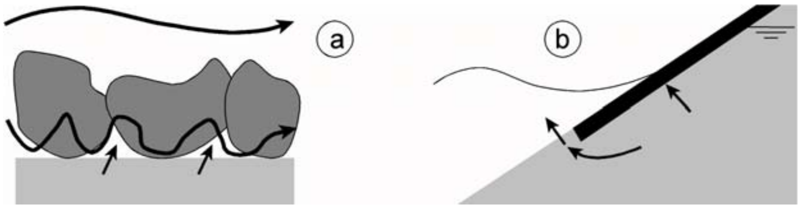

Figure 1-13 Ill-designed protections

In case (a), large rocks have been dumped on a sandy bottom which erodes because of currents. The rocks lead to a slightly lower velocity at the bottom, but to a considerable increase in turbulence and hence, maybe even an increase of erosion. Case (b) shows an asphalt-protection on a slope which would otherwise erode due to wave action. The protection now causes a difference between the water-tables inside and outside of the slope during low water. This head difference causes pressures on the protection which can result in lifting the protection layer. It also causes a concentrated groundwater-flow at the edge of the asphalt which leads to erosion at that spot.

Figure 1-14 shows the forces that act on a protected slope. A represents the loads from the water-side of the interface, the external load due to waves and currents. C is the load from inside due to a relatively high groundwater-potential in the soil-mass. B is the interaction between the external load and the inside of the structure. Although the external forces are usually rather violent and spectacular, many protections fail because of B or C.

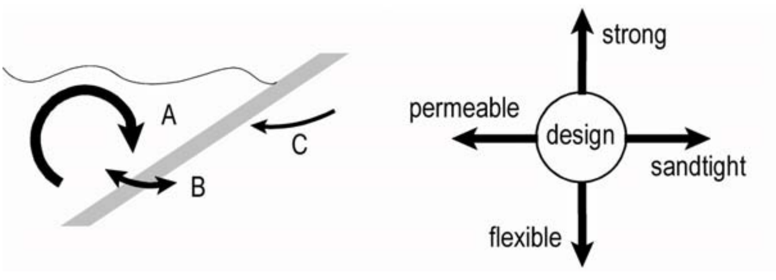

Figure 1-14 Contradicting demands

The external forces A require a *strong* protection. This strength can be obtained by using large, heavy stones. The example in Figure 1-13 has shown that a protection should also be *sand tight* due to process B. To make the protection sand tight, some layer is needed between the top-layer and the subsoil e.g. a filter, a cloth or a foil. But if that layer is impermeable, C can become a threat. That means *permeability* can be required (unless there are other reasons to make the protection impermeable; in that case the protection has to be designed to withstand the possible pressures). Another way of increasing strength is ensuring coherence in the top-layer e.g. by using concrete or asphalt instead of dumped rocks. The protection can then become impermeable or stiff which can cause problems if settlements are expected. So **f***lexibility* is another factor to reckon with. Figure 1-14 gives an idea of the contradicting factors in design of a protection.

Failure mechanisms

In general, it is always necessary to keep the overall picture in mind. Figure 1-15 shows the relevant failure mechanisms for a revetment.

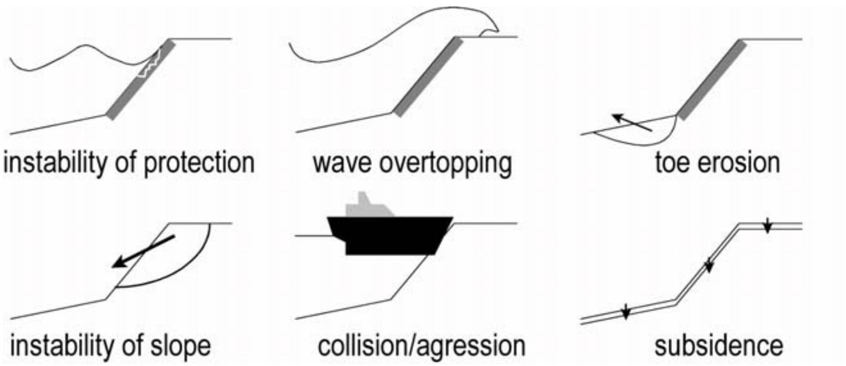

Figure 1-15 Failure mechanisms

Even if these mechanisms are not completely open to computation, a mere qualitative understanding can help to prevent an unbalanced design. Insight is more important than having an accurate formula to compute dimensions of some part of the structure. *Protections seldom fail because of an underestimation of the loads of 10 %; most protections fail because a mechanism has been neglected!*

Sometimes, designers put most of their energy into the first failure mechanism, the instability of the protection layer, also including the filter action (mechanism B in Figure 1-14). But if the protection is too low, wave overtopping can destroy the revetment. Toe protection is often neglected or underestimated. Instability of the slope can be of a micro or macro nature, both connected with the slope angle; Chapter 5 gives more details. Collision or aggression is self evident, but hard to include in a design. A collision proof design will be unnecessarily expensive, unless the protection is situated at a notorious accident spot. A better approach can be to keep repair material in stock and to anticipate repair works in the maintenance programme.

The failure mechanisms of a structure can be combined in a fault tree (Figure 1-16), which gives the relations between the possible causes and the failure of the revetment on top of the tree. If you are able to assign probabilities to the events, it is possible to determine the total probability of failure of the structure and to find weak spots. But also without that quantitative information it is useful to draw a fault tree to get the overall picture. An experienced designer does so intuitively but even then, it is a useful tool.

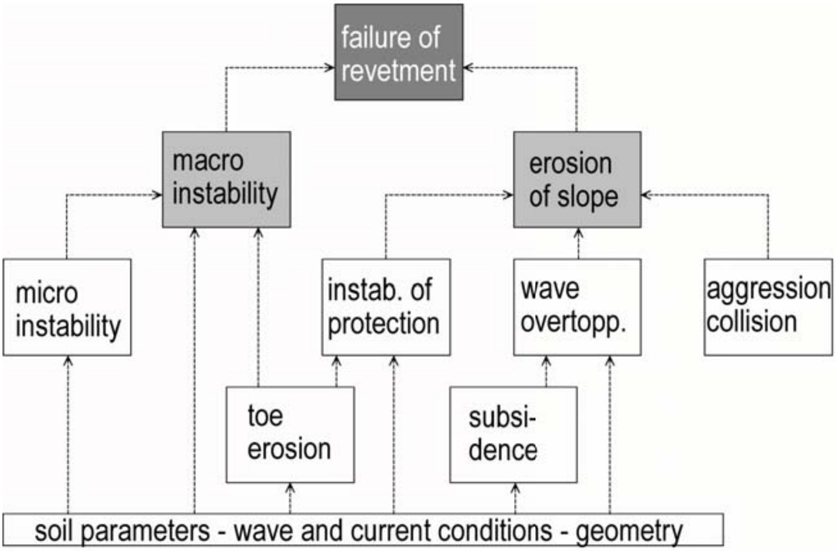

Figure 1-16 Fault tree

1.2.3 Load and strength

The core item in this book is the design of protections that can withstand the loads due to currents, water-level differences or waves. For structures consisting of relatively small elements (rocks, stones, blocks etc.) the definition of strength is somewhat ambiguous. A comparison with a steel structure is made to clarify this point (Figure 1-17).

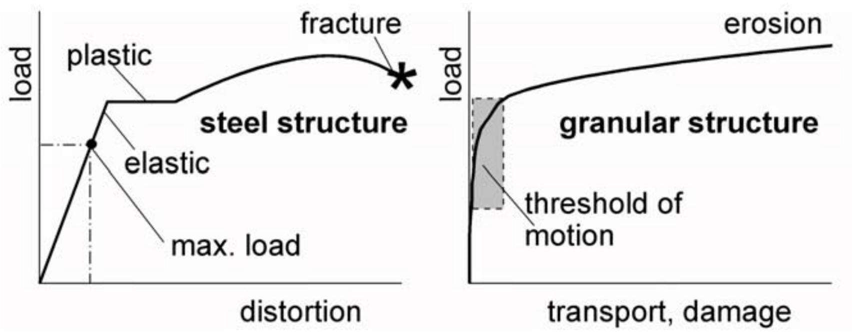

Figure 1-17 Load and strength for steel and granular structure

When steel is loaded, at first the distortion is elastic, obeying Hooke's law. At a certain point, without increase of the load, the distortion becomes plastic. After that, some strengthening occurs until the steel yields. The strength of steel is normally chosen well under the plastic limit, based on the statistics of the steel quality. The clear change of material behaviour serves as an indicator for the permissible strength

For a granular structure, things are less clear. When the load is small compared to the strength, nothing happens at all. At a certain load, some elements move and stop again after some time. Further increase of the load leads to more displacements, finally leading to complete erosion. Usually some damage, like the displacement of some stones on a slope, is not much of a problem; this also depends on the maintenance policy. A clear limit between acceptable and not-acceptable erosion is lacking and the threshold of motion has to be defined. This will be done for the various phenomena in the following chapters.

Another difference between steel and stones is that, for steel, both load and strength can be expressed in the same unit: Newton. For a protection this could be achieved by expressing the load on an individual stone in N and defining the strength as the weight of that stone (mass*g = N), but that is not very practical. It is customary to express the load in terms of the wave height or the current velocity. The strength is then indicated with a diameter or thickness, d, often as well as the relative density of the material [$\Delta = (\rho_m - \rho_w)/\ \rho_w$], which contributes to the strength. This leads to dimensionless parameters like $H/\Delta d$ or $u^2/\Delta gd$.

This can lead to confusion because, in hydraulic engineering, these dimensionless parameters are used both as *mobility* parameters and as *stability* parameters. The difference becomes clear when you consider the mobility parameter as an independent variable in a transport equation and the stability parameter as a dependent variable in a stability equation:

Transport, damage $= f$ (**mobility parameter**, geometry, etc)
Stability (parameter) $= f$ (accepted damage, geometry, etc)

The first type of equation includes many sediment transport equations, e.g. in this book the Paintal equation in Chapter 3 or, in a modified form, in scour relations in Chapter 4. Most relations in this book are of the second type, like those by Shields and Izbash in Chapter 3 or those by Van der Meer and Hudson in Chapter 8. It is good to be aware of the difference, as the use of these parameters in textbooks is not always consistent.

When used as a *mobility* parameter, a large value indicates more mobility (high load versus low strength). When used as a *stability* parameter, a larger value of $H/\Delta d$ indicates more stability (the same stone size can resist a larger wave or for the same wave, a smaller stone can be used). The *stability* parameter can be seen as a *critical value of the mobility* parameter, since the amount of acceptable damage or transport has been chosen. This may be confusing, but is essential in working with the different formulas.

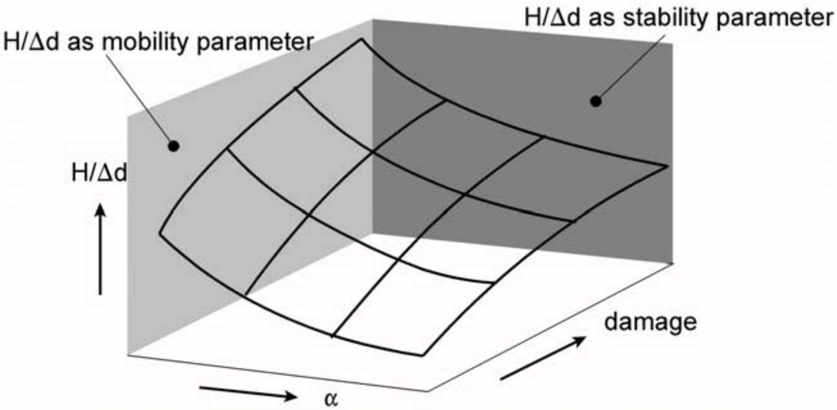

Figure 1-18 Mobility versus stability

The difference can be illustrated by the stability of stones on a slope in breaking waves (See Figure 1-18) based on the Van der Meer relations, see Chapter 8. In the $H/\Delta d$ - damage plane, $H/\Delta d$ is a *mobility* parameter, in the $H/\Delta d - \alpha$ plane it is a *stability* parameter. For a certain slope angle, α, and given stone dimensions (Δd), a higher wave, hence a greater $H/\Delta d$, gives more mobility or more damage. For a certain acceptable damage, a given stone (Δd) can stand higher waves when the slope is gentler (smaller α). This has to do with gravity, which reduces the strength of a stone on a steep slope, but also with the different behaviour of breaking waves on different slope angles, resulting in different loads. This, again, illustrates that load and strength are not defined unambiguously the way they are for a steel structure, when both can be expressed in Newtons.

Example 1-1

A beach coast with a wave height of 3 m during a storm and sand with a grain size of 0.2 mm gives a $H/\Delta d$ of about 10000. During that storm, a lot of sand will be transported. The same wave height with a concrete caisson wall will give an $H/\Delta d$ of less than 1 and no movement at all. So, a higher value of the mobility parameter indicates less stability.
The same coast is going to be protected with stones on a slope. From experiments it is found that the stability parameter for a slope 1:2 is about 2 (with hardly any movement of stones) and for a slope 1:4 is about 2.5 (with the same degree of damage). With the given wave height in a design storm of 3 m, this would lead to a stone size of 0.9 m for the 1:2 slope and 0.7 for the 1:4 slope. So, a higher value of the stability parameter indicates more stability.

Load and strength as design options

When the load exceeds the strength and measures have to be taken, there are two possible approaches: the strength can be increased or the load can be reduced. Figure 1-19 illustrates these possibilities. A bank, eroding due to wave action, can be protected by making a revetment (case A) or by constructing a wave reductor in front of the bank (case B). The latter can be chosen when the "natural" look of the bank has to be preserved, see also Chapter 12.

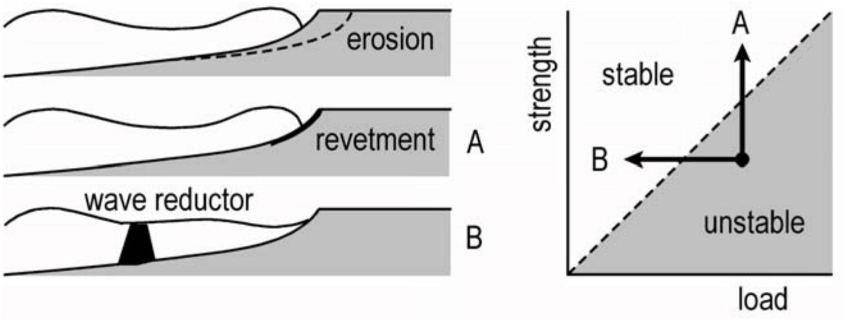

Figure 1-19 Strength increase or load reduction

Load and strength statistics

Loads in nature show a lot of variation. Waves depend on wind, velocities in a river depend on rainfall, so loads heavily depend on meteorology which has a random character. Probabilistic design methods, therefore, are important for protections. In a feasibility study it is often sufficient to work with a representative load. The choice of that load should be based on the relevant failure mechanisms (see Figure 1-15) and on the consequences of exceeding the load. Stability of the top layer of a bed protection behind a sluice (e.g. see Figure 1-3) is mainly sensitive for exceptional loads and should therefore be based on an extreme event, while erosion behind the protection is also determined by everyday flow. The use of different exceedance frequencies results in different design values for the same loading phenomenon!

More in general, the performance of a structure should be judged under various circumstances related to different limit states, see *e.g.* Vrijling et al, 1992. Two widely used limit states are:

Ultimate limit state (ULS): This limit state defines collapse or such deformation that the structure as a whole can no longer perform its main task. It is usually related to extreme load conditions. Related to the levels of Section 1.1.2, it can mean e.g. the collapse of a dike (meso level). In the fault tree in Figure 1-16 the ULS is represented by the higher part of the tree.

Serviceability limit state (SLS): This limit state defines the required performance, e.g. the wave reduction by breakwaters in a harbour. In the context of this book it describes a state that needs to be maintained. Related to the levels of Section 1.1.2 it means e.g. the damage of a dike protection (mini level).
Note: for the top layer of the protection this could be seen as the ULS to show that these definitions also depend on the level of detail. In Figure 1-16 the SLS is related to the bottom part of the tree.

The accepted probability of reaching both limit states is a function of the damage caused by exceeding that state. It is obvious that the chance of reaching the ULS should be much lower than reaching the SLS. Maintenance policy is closely related to these limit states. The strength of the structure *as a whole* can drop below the level that is needed under extreme conditions (see Figure 1-20). As long as these conditions do not occur, the ULS will not be reached. When deterioration goes on for a long time, the strength can become too small even for daily conditions and collapse will occur out of the blue. The extreme load in Figure 1-20 has some probability. When the strength is greater than the extreme load, the probability of reaching the ULS is considered acceptable. Without maintenance, the strength decreases and the failure probability increases until it reaches about 100 % when the strength becomes lower than the ever present loads. See Chapter 10 for more detail.

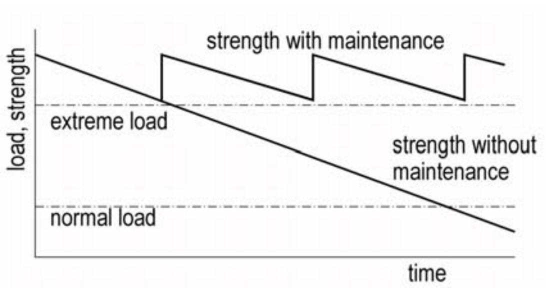

Figure 1-20 Strength as a function of time and maintenance

1.3 How to deal with this book

There are many textbooks on protection design. They are often aimed at professionals and deal with specific practical applications without treating the theoretical backgrounds. There are also many books on the theoretical backgrounds of flow and wave phenomena without practical application. This book aims to introduce protection design with a focus on the link between theory and practice. It is intended for use as a textbook in a graduate course on university level. The reader is supposed to be familiar with basic knowledge of hydraulics and soil mechanics; only the most important elements thereof will be treated in this book.

Some engineers are addicted to formulas and computing. Formulas are indispensable to calculate dimensions, but again, it is stressed that insight is often more important than numbers. There are many formulas in this book, and of course, they are meant to be used to calculate the dimensions of protections but there is also another way to look at them. Formulas are a very special form of language; they are the most concise way to express a phenomenon. By reading them carefully in this way, it is possible to gain some insight because they show the relations between different parameters, thus describing a phenomenon. The worst thing that can happen to a formula is to be learned by heart without being understood. Another accident that can happen with a formula is that it is considered algebra instead of physics. When doing so, cause and effect can be interchanged freely, sometimes with funny computational results.

The best way to read a formula is to start with the parameters. Do they seem logical for the process described, are any parameters missing? When a parameter's value doubles, what happens to the result and does that seem reasonable? What is the domain in which the formula is valid? Empirical relations are only valid in the range of experiments; theoretical formulae are often based on simplifications.

Another essential element in the book is made up of pictures. These too, present a concise language, either by means of "real life" pictures of some phenomenon or by means of graphs describing the relation between parameters. Text, formulae and pictures together tell the story of protecting the interface between soil and water. Interpretation of these three requires the ability to imagine what is happening to the water, the sediment and the structures. Keeping an open eye and mind when walking along a bank or coast, or in any other place where water moves, surely helps.

This introductory chapter tries to reveal the core of the whole subject and therefore also sometimes resembles a summary. The reader is advised to read it before and after studying this book. Much of what is not clear when reading this chapter the first time, might be recognized immediately when reading it again later. There is a saying: "*Understanding is nothing but getting used to*" which contains some truth.

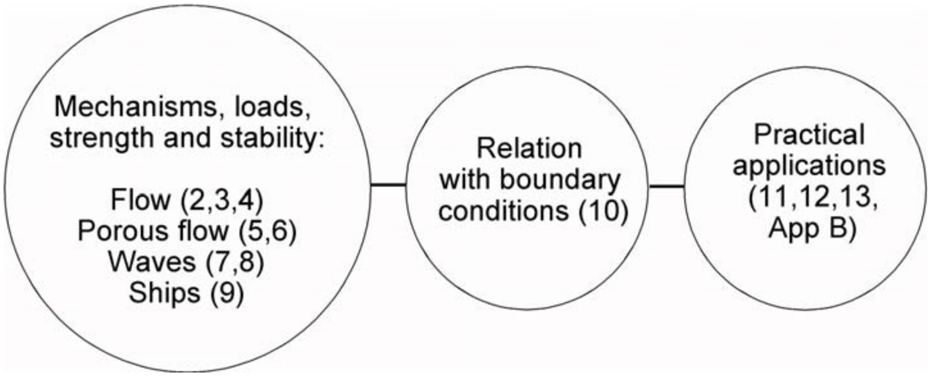

Figure 1-21 Main structure of this book

The contents of the rest of the book can be divided into a more theoretical part (Chapters 2–9, an application of theoretical hydrodynamics and soil mechanics) and a part that deals with the applications of protections (Chapters 10-13). Chapter 2 to 9 contain the technical heart of the matter and have a logical composition. It starts with flow phenomena and related erosion and stability problems (2, 3 and 4). Porous flow is the next step with a small addition concerning geotechnical issues (5 and 6). Wind wave phenomena are treated in Chapter 7 and related erosion and stability problems in Chapter 8. Chapter 9 deals with all aspects of erosion and stability related to ships.

This chapter refers to many of the previous chapters, since the ship-related phenomena contain both elements of flow and waves.

Chapter 10 is on dimensioning of protections and mainly deals with probabilistic methods. Chapter 11 contains examples of protections as made in several places in the world, the focus being on The Netherlands. Chapter 12 looks into environmental aspects, with a focus on nature-friendly protections. The construction of protection works is the subject of Chapter 13.

Consequently implementing the order of subjects appeared to be impossible. Filters e.g. are treated in Chapter 6, including filters under wave loads, while waves are not treated until Chapter 7. This has been done because a special section on filters in Chapter 7 appeared to become too insignificant. The alternative of treating porous flow and filters after waves was not attractive, as porous flow plays a role in the stability of block revetments.

The main text in each chapter contains the basic message of this book with a one-page *summary* at the end of the chapter. *Intermezzo's* sometimes clarify the main text or give some historic background. *Examples* are intended to illustrate the application of formulas. Some chapters have *Appendices*. These are, by definition, no part of the main text. They serve as background information on subjects that are supposed to have been studied but that have possibly been forgotten somewhat. The same is valid for *Reminder I* which contains simple equations that you should know by heart already, but the reminder comes in handy when you do not. *Reminder II* contains some interesting details about the contents of this book. They may sometimes be overlooked easily in the avalanche of information, but can come in handy when one is confronted with a protection problem.

There are two general appendices: A and B. Appendix A gives information on materials, to be used in the many formulas in the book. Appendix B gives some elaborated example cases.

Finally, the *References* are for those who think that not everything is in this book, which is indeed the case.

2 FLOW – Loads

(Oosterschelde barrier, photo Rijkswaterstaat)

2.1 Introduction

When designing a protection it is necessary to have rather detailed information about the velocity field. For many projects, flow data is available from historical records or from a network model which calculates overall quantities, like discharges and hence, given the geometry of the situation, as average velocities: $\bar{u} = Q/A$.

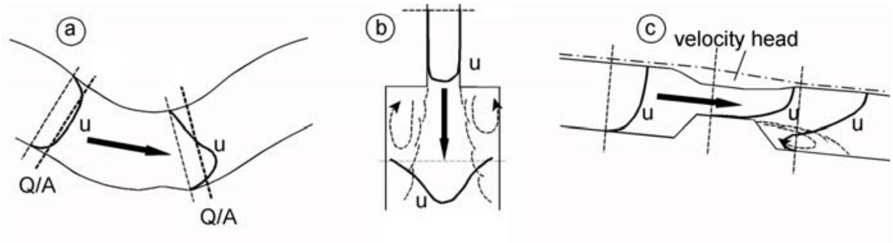

Figure 2-1 Velocity field in various situations

For the design of a bank protection in a river bend (see Figure 2-1a), the velocity near the bank must be known, which can be deducted from measurements in the river or in a scale model, from a numerical model or from a sketch, based on some understanding of the flow. In Figure 2-1b, it is inappropriate to work with a velocity averaged over the cross-section downstream of the outflow, since the flow direction in some parts is opposite to the main flow direction due to separation between the main flow and the flow near the side-walls. Figure 2-1c shows a similar situation in a vertical cross-section. Working with averaged velocity values ($\bar{u} = Q/A$), e.g. with Chezy's law for uniform flow: $\bar{u} = C\sqrt{Ri}$, would produce nearly the same value for the velocity upstream and downstream of the sill, since the geometry is the same. The figure, however, shows a completely different flow situation. Upstream, the velocity is well represented with a logarithmic profile, as can be expected in a (stationary) uniform flow. Downstream of the sill there is a flow separation with an eddy in which the flow direction near the bottom is opposite to the mainstream. The eddy can be seen as an ill-defined boundary for the flow, which also influences the turbulence level. In practice, the flow is always turbulent, but at the transition between main flow and eddies, the turbulence will be much greater and will persevere, far downstream from the transition. Protections of the interface between water and soil in these areas need to be relatively strong, because without protection, the erosion will be considerable.

Understanding the way water flows is paramount for every hydraulic engineer and a sketch of the velocity field should mark the start of every project. For such a sketch it is necessary to go from average values of the velocity to a local value and requires some understanding of the turbulence. In general one can say that it is necessary to have some insight in what is happening *inside* the water. Although turbulence is one of the most complex subjects in hydraulics, some basic facets of turbulence will be reviewed in the following section, focussing on phenomena rather than on formulas.

2.2 Turbulence

For the physical background of turbulence see Appendix 2.8.2. This section discusses the characterization and importance of turbulence in hydraulic engineering. One of the most striking features of turbulent motion is that the velocity and pressure show irregular fluctuations, see Figure 2-2.

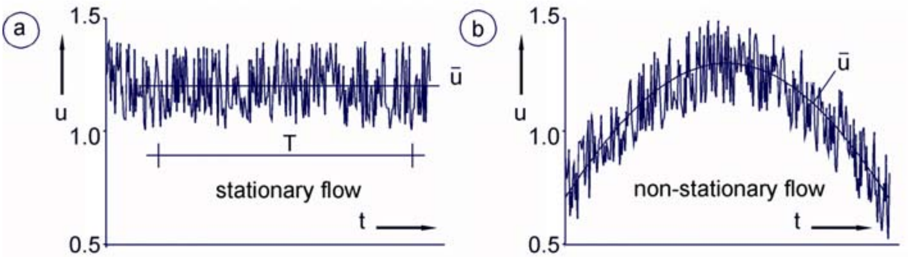

Figure 2-2 Velocity registration in turbulent flow

The abundance of definitions of turbulence indicates that the subject is complicated. The definition according to Hinze, 1975 is: "*Turbulent fluid motion is an irregular motion, but statistically distinct average values can be discerned and can be described by laws of probability*". To do so, velocities are averaged over a certain period of time, "smoothing" out the turbulent fluctuations. The values of velocity and pressure can be written as:

$$u = \overline{u} + u' \quad v = \overline{v} + v' \quad w = \overline{w} + w' \quad p = \overline{p} + p' \tag{2.1}$$

in which ¯ indicates the average value and ´ a measure of the fluctuations. The averaging period T (see Figure 2-2a), sometimes called the *turbulence period*, is chosen such that it is long enough to smooth out turbulence and short compared with the principal motion. Figure 2-2a is valid for a *stationary* flow, where stationary means that the *average* is constant in time. A turbulent signal in a non-stationary flow can be smoothed out, using a moving average, see Figure 2-2b.

The next step in obtaining *statistically distinct average values* is defining a measure of the intensity of the velocity fluctuations. This can not be done directly with the averages of the fluctuations, since these are 0 by definition, see Figure 2-2. Therefore the squares of the fluctuations are used and are averaged. The intensity is now defined as the square root of this average and again has the same dimension as a velocity: the root-mean-square value (r.m.s., which is equal to the standard deviation in a Gaussian distributed signal).
Turbulence can then be expressed in various ways, such as:

$$k = \frac{1}{2}\left(\overline{u'^2} + \overline{v'^2} + \overline{w'^2}\right), \quad r_u = \frac{\sqrt{\overline{u'^2}}}{\overline{u}}, r_v = \frac{\sqrt{\overline{v'^2}}}{\overline{u}}, r_w = \frac{\sqrt{\overline{w'^2}}}{\overline{u}} \tag{2.2}$$

k represents the total kinetic energy in a turbulent flow, r the relative fluctuation intensities of u, v and w, all of them compared with the main flow component, which is customary. When r is used without any index, it means r_u. One should always be aware of the parameter that is used to make the rms-value dimensionless. It is possible to relate the turbulent fluctuations (u') to an average value ($\bar{u}$) measured in a different location.

Be aware of the fact that r becomes a rather useless parameter when the average velocity ($\bar{u}$) is small in respect to the fluctuations (u').

Reynolds stresses

Appendix 2.8.1, on basic equations, shows that, in a 2-dimensional situation, extra normal and shear stresses appear in the momentum equation, due to the use of an average velocity:

$$\underbrace{\underbrace{\rho\left(\frac{\partial \bar{u}}{\partial t} + \bar{u}\frac{\partial \bar{u}}{\partial x} + \bar{w}\frac{\partial \bar{u}}{\partial z}\right)}_{\text{inertia}} = \underbrace{-\frac{\partial \bar{p}}{\partial x}}_{\text{press.}} + \underbrace{\mu\frac{\partial^2 \bar{u}}{\partial z^2}}_{\text{visc.}}}_{\text{mean values}} \underbrace{- \rho\left(\frac{\partial \overline{u'^2}}{\partial x} + \frac{\partial \overline{u'w'}}{\partial z}\right)}_{\text{Reynolds stresses; turb. fluctuations}} \tag{2.3}$$

These extra terms originate from the non-linear convective inertia terms and have the same dimension as stresses, but how logical is it to consider them as stresses? The following is a qualitative analogy with elementary mechanics (adapted from LeMéhauté, 1976).

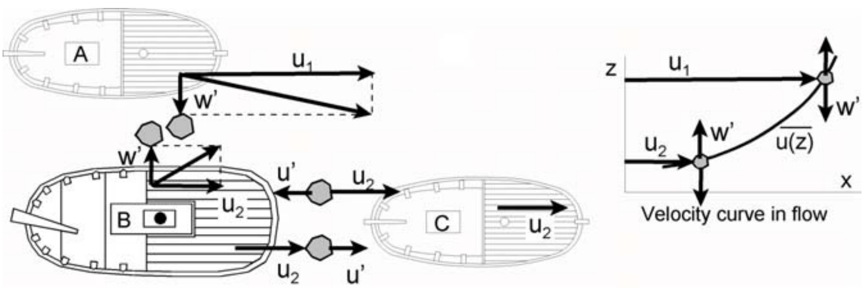

Figure 2-3 Exchange of momentum due to turbulence

Consider the quarrelling fishermen in Figure 2-3. A and B throw stones with mass m at each other with relative speed w'. When they hit the target, with the vectorially added velocities w' and u_1 or u_2 respectively, the stone from A accelerates B, because

$u_1 > u_2$ and, reversely, the stone from B decelerates A. The exchange of momentum is equal to $m(u_1 - u_2)$, negative for A and positive for B.

Another analogy could be found in people walking in a very crowded street where those on the sidewalk walk slower than those in the middle of the street. When being pushed from the sidewalk, a pedestrian will be accelerated by the faster moving crowd and vice versa.

A similar situation exists in flowing water with a velocity gradient, see the velocity curve in Figure 2-3. Consider a mean flow with $\bar{u} = \bar{u}(z)$, and $\mathrm{d}\bar{u}/\mathrm{d}z > 0$. The particles that travel upwards arrive at a layer with a higher velocity $\bar{u}$. These particles (or lumps) preserve their original velocity causing a negative component u', thus decelerating the flow in x-direction. Conversely, the particles arriving from above give rise to a positive u', thus accelerating the flow. The flux of mass (per unit of area and per unit of time) between the layers is equal to $\rho\, w'$ (compare the masses of the stones thrown from the ships in Figure 2-3), hence the transferred momentum = $\rho w'(u_1 - u_2)$ which looks similar to $\rho\, w'u'$ (with u' as a measure of $u_1 - u_2$). The vertical flux of turbulent momentum per unit surface area is then equivalent to a stress:

$$M_z = \rho \cdot u' \cdot w' = \tau_{xz} \left(= \frac{\mathrm{kg}}{\mathrm{m}^3} \frac{\mathrm{m}}{\mathrm{s}} \frac{\mathrm{m}}{\mathrm{s}} = \frac{\mathrm{kg}\,\mathrm{m}}{\mathrm{s}^2 \mathrm{m}^2} = \frac{\mathrm{N}}{\mathrm{m}^2} \right) \tag{2.4}$$

Similarly, one can see the analogy between the Reynolds normal stress ($\rho\, u'u'$) and stones thrown between B and C in the figure. Both stones result in a net force to the right between B and C. Due to this stress, the force acting on a motionless body in a current with a certain velocity can be 10% greater than the force acting on an identical body dragged with the same velocity through water at rest.

Flow resistance

In laminar flow the resistance, expressed as a shear stress, is proportional with the flow velocity. In turbulent flow, the quadratic terms in Equation (2.3) become dominant and the relation between τ and u becomes quadratic, see Figure 2-4a:

$$\tau = c_f \rho u^2 \tag{2.5}$$

Figure 2-4b shows the velocity distribution in a pipe or channel for laminar and turbulent flow. The velocity is more homogeneous in a turbulent flow, as a result of the turbulent exchange of momentum.

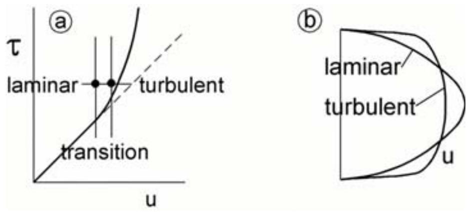

Figure 2-4 Resistance in laminar and turbulent flow

Intermezzo 2-1

The Reynolds stresses appear in Equation (2.3) due to the use of average velocities, see Appendix 2.8.1. But how can extra forces be created just because a mathematical procedure is followed? And who informs the water of these extra forces? Of course, averaging cannot create forces and water flows nicely, even without our equations.

When the flow is laminar, the fluctuations in Equation (2.3) are equal to 0 and there are no Reynolds stresses. When the flow becomes turbulent, the fluctuations in Figure 2-2a create extra forces as was deducted using Figure 2-3. Since we use ū, both in laminar and turbulent flow, we would omit these extra forces without the Reynold stresses.

2.3 Wall flow

2.3.1 Uniform flow

The most elementary and one of the best known cases of wall flow is uniform flow in a channel. All of the phases of the energy cascade in Section 2.8.2 are present in every cross-section of the flow. The slope causes a continuous transformation of potential energy, via kinetic energy in the main flow and in the turbulent eddies, into heat. An equilibrium exists between the bottom shear stress and the component of the fluid pressure on the slope, see Figure 2-5.

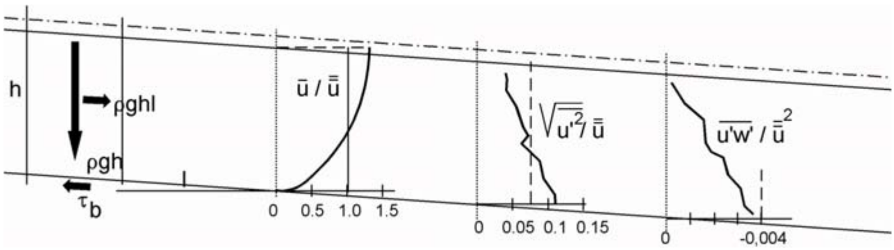

Figure 2-5 Uniform flow

The bottom shear stress is also related to the velocity, see Equation (2.5). The velocity averaged over the height and in time is used (two overbars, in everyday use often reduced to one, or even none). This leads to:

$$\tau_b = \rho g h I = c_f \, \rho \, \bar{\bar{u}}^2 (= \rho u_*^2 = \rho \overline{u_b' w_b'}) \quad \Rightarrow \quad \bar{\bar{u}} = \frac{1}{\sqrt{c_f}} \sqrt{g h I} \qquad (2.6)$$

The use of h is only correct for an infinitely wide channel; otherwise the hydraulic radius R should be used; c_f is a dimensionless friction coefficient. In this equation also the so-called shear velocity u_* is introduced:

$$u_* = \sqrt{\tau / \rho} \tag{2.7}$$

The shear velocity is a parameter with the dimension of velocity, but in fact it expresses the shear stress; it is not a velocity which can be measured in a physical environment.

In traditional hydraulics, empirical relations are used, such as:

$$\begin{aligned} &\text{Chezy:} \quad \bar{\bar{u}} = C\sqrt{RI} && \text{with: } C = \sqrt{\frac{g}{c_f}} \\ &\text{Manning:} \quad \bar{\bar{u}} = \frac{1}{n} R^{2/3} \sqrt{I} && \text{with: } n = R^{1/6}\sqrt{\frac{c_f}{g}} \end{aligned} \tag{2.8}$$

The Chezy coefficient C and the Manning number n are not dimensionless; both definitions contain the acceleration of gravity, g. So gravity, which is responsible for the flow, is only implicitly present in these empirical equations! C is expressed in [$m^{1/2}/s$] and n in [$s/m^{1/3}$]. n is a roughness coefficient, while C is actually a "smoothness" coefficient (a large value of C means little roughness). From Equations (2.6) and (2.8) together with the definition of u_* follows:

$$u_* = \bar{\bar{u}}\sqrt{g} / C \tag{2.9}$$

C, which is used in this book, can be related to the so-called equivalent sand roughness according to Nikuradse-Colebrook, for a hydraulically rough situation as follows:

$$C = \frac{\sqrt{g}}{\kappa} \ln \frac{12\,R}{k_r} \approx 18 \log \frac{12\,R}{k_r} \qquad (k_r \text{ is equivalent roughness}) \tag{2.10}$$

For a smooth bed with grains, the Nikuradse roughness k_r usually equals several times the characteristic grain diameter (see Chapter 3). For a moving bed, higher values are possible due to the formation of ripples or dunes.

Figure 2-5 also gives some measurements in a uniform flow. One bar over u means averaged over the turbulence period and the double bar means averaged over the turbulence period and over the waterdepth. For practical reasons, this notation will not be used consequently in the following sections. The vertical velocity profile is logarithmic with an average velocity at about 0.4 times the waterdepth from the bottom. The turbulent fluctuations can be approximated with:

$$\overline{r} = \frac{1}{\overline{\overline{u}}\,h}\int_0^h \sqrt{\overline{u'^2}(z)}\,dz = 1.2\frac{\sqrt{g}}{C} \qquad \frac{\overline{u_b' w_b'}}{\overline{\overline{u}}^2} = \frac{g}{C^2} \tag{2.11}$$

The expression for the depth-averaged fluctuation was derived from numerical computations by Hoffmans, 1993, while the turbulent shear stress follows directly from equation (2.6) and (2.8). Now that we have simple equations that express the turbulent quantities, we can calculate the Reynolds-stresses. In uniform wall flow these are a function of the roughness only. This is not really surprising, as the wall roughness is the only source of turbulence in wall flow, which is also clearly visible in the measurements in Figure 2-5. C-values are normally in the range 40 to 60 √m/s, giving depth-averaged values of r of 0.06 to 0.1

Example 2-1

20 m^3/s of water flows in a 10 m wide channel with vertical banks, a bed slope of 1/1000 and a roughness of 0.2 m. What is the depth, the velocity, the Chezy-value, the relative turbulence intensity and the relative turbulent shear stress?

Calculating the depth and velocity, using the Chezy equation, is an iterative process. First, a depth of 1 m is estimated (any estimate will work). The hydraulic radius then becomes: $R = bh/(b+2h) = 10/22 = 0.83$ m. The Chezy-value becomes $18\log(12R/k_r) = 18\log(12\times0.83/0.2) = 30.6$ √m/s. Using $u = C\sqrt{RI}$, $u = 30.6\sqrt{0.83\times0.001} = 0.883$ m/s follows. The discharge would then be: $Q = bhu = 8.83$ m^3/s which is more than twice as little as the given discharge of 20 m^3/s. A new estimate of $h = 2$ m, with the same procedure gives $C = 34.8$ √m/s, $u = 1.32$ m/s and $Q = 26.3$ m^3/s, so, $h = 2$ m is too much and a lower value has to be chosen. The final result for $Q = 20$ m^3/s is: $h = 1.67$ m, $C = 33.8$ √m/s and $u = 1.2$ m/s.

Note: with sloping banks the iteration involves more parameters.

The relative turbulence intensity is approached with Equation (2.9): $r = 1.2\sqrt{g}/C = 0.11$ and the relative turbulent shear stress is: $g/C^2 = 8.6\ 10^{-3}$.

2.3.2 Non-uniform flow

In practice, flow is never uniform. Accelerations and decelerations influence the boundary layer and the turbulence in the flow. The boundary layer is defined as the region which is influenced by the presence of the wall, in contrast to the region outside the boundary layer. In stationary, uniform flow, the boundary layer is fully developed and takes up the entire water depth, leading to the logarithmic velocity distribution, see Figure 2-5. The following illustrations show wall flow with boundary layers that are not fully developed. When water suddenly flows along a wall, a boundary layer will start to grow, see e.g. Schlichting, 1968. This is illustrated in

Figure 2-6, showing the growth of a boundary layer when an infinitely thin plate is placed in a flow with $u = u_0$.

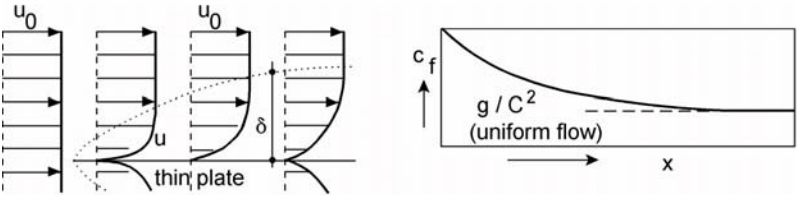

Figure 2-6 Growth of boundary layer

The shear stress along the plate slows down the flow and the exchange of momentum will lead to the growth of the boundary layer. This growth can be estimated roughly with $\delta(x) \approx 0.02\,x$ to $0.03\,x$, indicating that after a distance equal to 30 - 50 times the waterdepth, the flow will be fully developed and the boundary layer will take up the entire waterdepth. With the growth of δ, the shear stress decreases and eventually, c_f reaches the value expressed in Equation (2.6).

In the situation in Figure 2-6 there are no accelerations or decelerations on a macro level; the thin plate only creates a new boundary layer. With accelerations and decelerations, another situation arises. Acceleration is due to a pressure gradient in the flow direction and an opposite gradient leads to deceleration, both cause a change in the boundary layer thickness. The change in δ is roughly given by (see Booij, 1992):

$$\frac{d\delta}{dx} = \frac{-\,(\,4 \text{ to } 5\,)\,\delta}{u_0}\,\frac{du_0}{dx} \tag{2.12}$$

indicating that acceleration causes reduction of the boundary layer thickness, while deceleration does the opposite. These changes of the boundary layer are visible in the velocity distribution, see Figure 2-7. In accelerating flow, the velocity profile becomes fuller, increasing $\partial u/\partial z$ and hence increasing the shear stress.

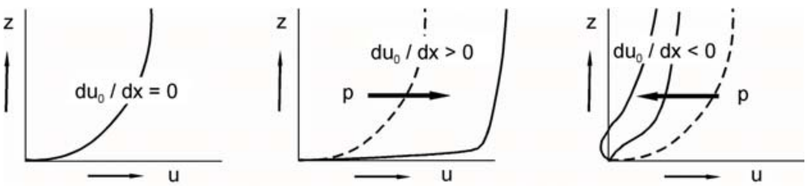

Figure 2-7 Influence of pressure gradient on velocity profile

The effect of acceleration on turbulence can be seen in Figure 2-8. In a wind tunnel, turbulence is created by means of a grid. The fluctuations decrease downstream from the grid. In the contraction, the fluctuations in the flow direction (u') decrease even further, because the flow concentrates. The fluctuations perpendicular to the flow increase in the contraction. The total amount of turbulent kinetic energy, k, remains approximately constant. Due to the increased velocity in the contraction, the relative turbulence, r, using the *local* mean velocity, see Equation (2.2), decreases.

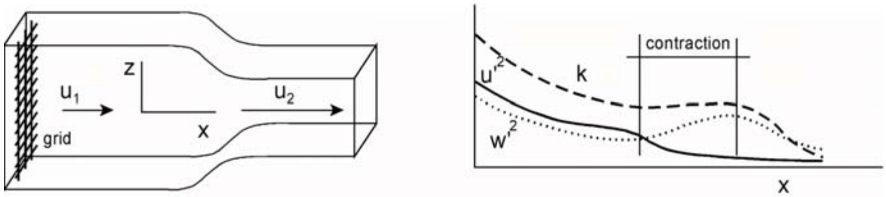

Figure 2-8 Turbulence in wind tunnel contraction (from: Reynolds, 1977)

In decelerating flow the opposite happens: growth of the boundary layer with possibly flow separation, see Figure 2-7 and considerable increase in turbulence. The following section show several examples.

2.4 Free flow

2.4.1 Mixing layers

Whenever two bodies of fluid move along each other with different velocities, a mixing layer will grow between them, comparable with the growth of the boundary layer in wall flow. Figure 2-9 shows some characteristics of mixing layers (from Rajaratnam, 1976). In this figure one layer flows and the other is stagnant for reasons of simplicity. This is not essential; any velocity difference causes a mixing layer. The stagnant fluid will accelerate, whereas the flowing mass will lose momentum. In the mixing layer the shear stress is intense, inducing turbulence.

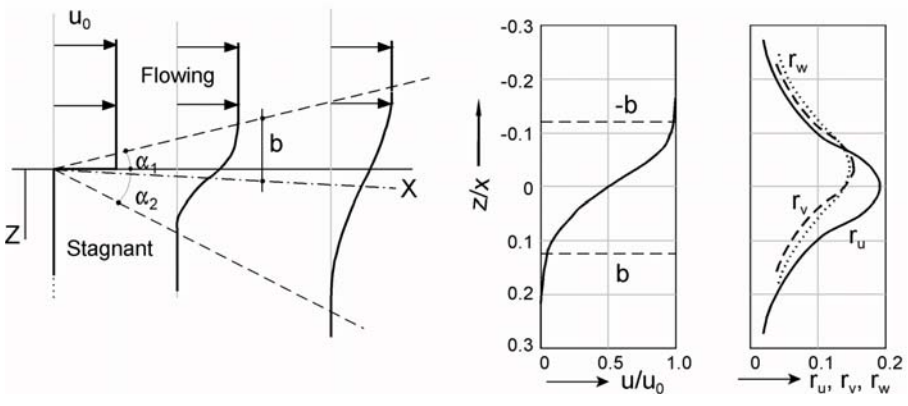

Figure 2-9 Flow, velocities and turbulence in mixing layer

The diverging angles of the mixing layer (see Figure 2-9) are $\alpha_1 \approx 5°$ and $\alpha_2 \approx 10°$, meaning that the flow penetrates into the stagnant area with a slope of 1:6. The line $u = u_0/2$ is taken as the x-axis, which deviates slightly in the direction of the stagnant zone. The width of the mixing layer, b, is defined as the distance between the points where the velocity is 0.99 u_0 and 0.5 u_0. The momentum equation shows that $b \propto x$ and from that we can assume similarity in the flow profiles, hence u/u_0 is a function of z/b only. The assumption of similarity has been confirmed in experiments, see Figure 2-9 which is based on measurements, see Rajaratnam, 1976. u/u_0 has been plotted against z/x, which is, with $b \propto x$, equivalent to z/b.

The velocity distribution in the mixing layer can be described reasonably with:

$$u = u_0 \ \text{ for } \ z < -b \quad \text{and} \quad u = u_0 \, e^{-0.693\left(\frac{z}{b}+1\right)^2} \ \text{ for } z \geq -b \ \text{ with } \ b \approx 0.12\,x \tag{2.13}$$

Since we are interested in turbulent velocity fluctuations, which influence stability and erosion, some measurements of r are also presented in Figure 2-9. Note that the fluctuations in all directions have been made dimensionless by using the driving velocity of the shear layer: u_0. The maximum of u' occurs where the velocity gradient is maximum and where u = 0.5 u_0. Compared with the local velocity, the velocity fluctuations are twice as high. The total turbulent kinetic energy in the centre of the mixing layer, related to u_0, is approximately: $k/u_0^2 = (r_u^2 + r_v^2 + r_w^2)/2 \approx$ ½ × $(0.2^2 + 0.15^2 + 0.15^2) \cdot 0.045$, see Equation (2.2).

2.4.2 Jets

A jet flows into a large body of water with a surplus velocity compared with the ambient fluid. Many flow situations in hydraulic engineering show resemblance to jets, e.g. flow from a culvert or the flow behind a ship's propeller. The falling water mass in a plunging breaker (see Chapter 7) also has jet-like features. A plane jet is a two-dimensional outflow from an orifice and a circular jet is 3-dimensional with axial symmetry. The jet attracts water from the ambient fluid, causing an increase the discharge in the flow direction with a constant momentum flux. The extremes are at one end an infinitely small orifice with an infinitely high velocity and at the other end an infinitely large flow area with an infinitely small velocity, but with the same momentum flux. Figure 2-10 shows some characteristics of jets.

Close to the orifice (the flow development region) turbulence penetrates into the core of the jet. This is the mixing layer as described in the previous section. The velocity in the centre-line is equal to u_0. After the flow has developed fully, the velocity decreases, the jet spreads and the dimensionless velocity profile no longer changes with x, see Figure 2-10.

Only results relevant to hydraulic engineering are presented. For a detailed description of jet phenomena, the reader is referred to Rajaratnam, 1976. The velocity distribution can be described as a Gaussian curve with only two parameters, u_m (u in the centre of the jet) and b (a typical width, usually defined where $u = u_m/2$). Plots of u/u_m against z/b (for plane jets; for circular jets against R/b, R being the radial distance from the centre) are similar for all x (in the developed region!, see

Figure 2-10). For plane jets: $u_m \propto 1/\sqrt{x}$ and $b \propto x$ can be derived and for circular jets: $u_m \propto 1/x$ and $b \propto x$. Numerical values for these proportionalities must be determined by experiments. For free jets the following proportionalities were found, see also Figure 2-10:

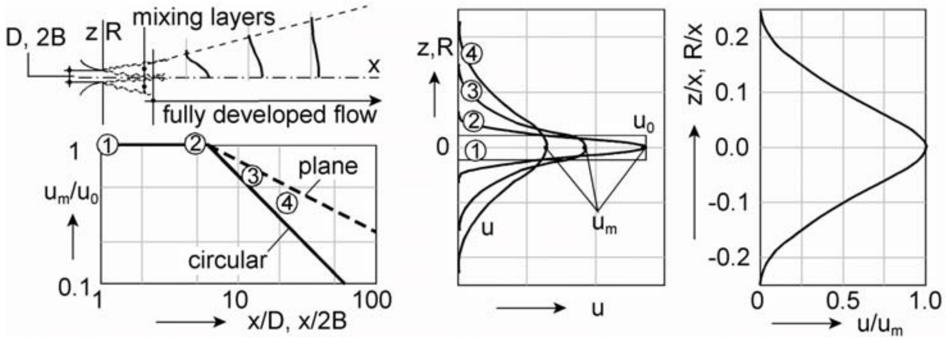

Figure 2-10 Flow and velocities in jets

$$\text{Plane jets: } u_m = \frac{3.5\, u_0}{\sqrt{x/B}} \qquad b = 0.1\,x \qquad u = u_m\, e^{\left(-0.693\left(\frac{z}{b}\right)^2\right)}$$

$$\text{Circular jets: } u_m = \frac{6.3\, u_0}{x/D} \qquad b = 0.1\,x \qquad u = u_m\, e^{\left(-0.693\left(\frac{R}{b}\right)^2\right)} \tag{2.14}$$

These expressions are only valid in the region of fully developed flow, which starts at about $x = 12B$ for plane jets (B is half the width of the orifice) and $x = 6D$ for circular jets (D is the radius of the orifice). In the flow development region the velocity can be approximated with u_0.

Since we are again interested in turbulent fluctuations, Figure 2-11 shows some measurements for a circular jet: Figure 2-11a in the centre-line and Figure 2-11b from the centre to the edges of the jet. Note that the fluctuations in all directions are relative to the velocity in x-direction, namely with u_m, which is the maximum velocity in the centre-line, which decreases proportionally with x. The relative fluctuations in the centre-line become constant with increasing x, reaching a value of about 30% in the centre-line for r_u. For R or $z = b \approx 0.1x$ the velocity is, by definition, $u_m/2$. The fluctuation there $(R/x = 0.1x/x = 0.1)$ is about 25% relative to u_m, so, relative to the local velocity, it is about 50%.
The above equations are for normal jets. A special case of a jet is the flow from a propeller of a ship. Because of the propeller, the turbulence in such a flow is much more, and differently distributed 1 than in a normal jet. This will be discussed in more detail in Chapter 9.

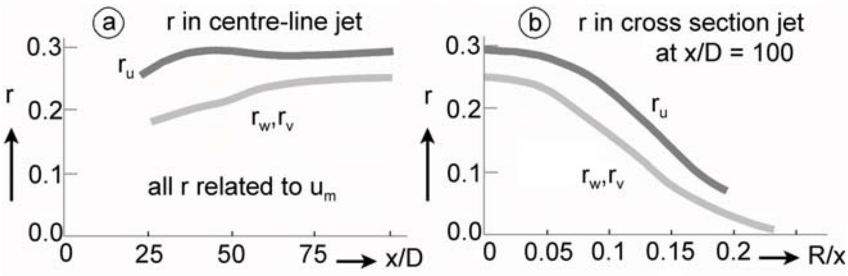

Figure 2-11 Turbulent fluctuations in circular jet

Example 2-2

A circular jet flows with 6 m/s from a nozzle with a diameter of 1.5 m. What is the velocity at a location 15 m downstream from the nozzle and 1 m from the axis of the jet?
From equation 2.13 the velocity in the jet axis 15 m downstream follows: $u_m = 6.3u_0/(x/D) = 6.3\times6/10 = 3.78$ m/s.
The velocity 1 m from the axis is: $u = u_m\exp(-0.693(R/0.1x)^2 = 2.78$ m/s.

2.5 Combination of wall flow and free flow

2.5.1 Flow separation

Most flow situations are combinations or alternations of wall flow and free flow. Examples are flow along sills, abutments, groynes or bridge piers but also constrictions and expansions in a river or canal. For the archetypes of geometrical variations (vertical and horizontal constrictions and expansions, detached bodies), acceleration, deceleration and turbulence are discussed and illustrated with experimental results. Again, the focus is on phenomena rather than on formulas.

In these situations both boundary layers and mixing layers are present. Flow separation can be seen as the transition from wall flow to free flow. In the case of a sharp edge (see Figure 2-12a) it is easy to see that the flow has to separate, since no separation would mean that a streamline would make a right angle with a radius $R = 0$, leading to an infinite centripetal force ($F_c = u^2/R$).

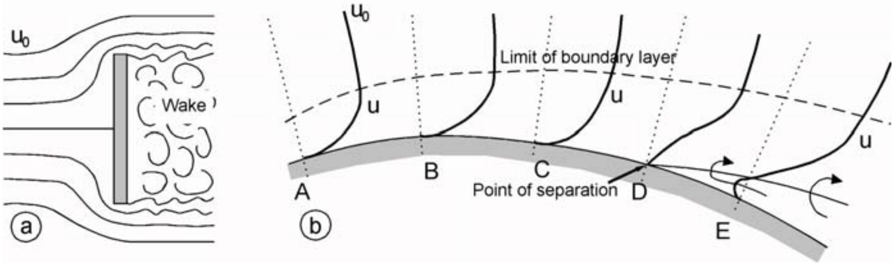

Figure 2-12 Flow separation around blunt and round body

So, the water cannot follow such a sharp curve and goes straight on. Here, the relation between (convective) inertia and viscosity plays a role. With (very) low *Re*-numbers, the flow can make the turn. Viscosity can then be seen as the "tension strength" of a fluid. When there is no fixed separation point, the situation is like in Figure 2-12b. From B onwards, the velocity decreases causing growth of the boundary layer up till point D (see also Figure 2-7 and Equation (2.12)) and eventually flow separation. This point of separation depends on the shape, *Re*-number and roughness of the body surface and can not be predicted accurately for an arbitrary body. Flow retardation and separation usually increases turbulence and hence the load caused by the flow.

2.5.2 Vertical expansion (backward-facing step)

A sudden increase in depth in the flow direction is often indicated as a backward facing step. The uniform wall flow before the step becomes free with a mixing layer immediately downstream of the step and eventually becomes uniform wall flow again with an increased depth. Figure 2-13 shows some measurements near a backward-facing step, mainly from Nakagawa and Nezu, 1987. Figure 2-13a shows the flow situation with some characteristic areas. Between the mixing layer and the bottom, a recirculation zone exists, which is absent in a completely free mixing layer. The location where the main flow "touches down" is the reattachment point. From there a new wall boundary layer grows and the flow tends to reach a new equilibrium situation. Figure 2-13b, c and d show the vertical distributions of $\bar{u}\ \bar{u}_1$, r_u (also related to $\bar{u}_1$) and $u'w'/\bar{u}_1^{\,2}$, respectively. $\bar{u}_1$ is the maximum (time averaged) velocity on top of the step. The mixing layer and the tendency to revert to uniform wall flow are clearly visible. The velocity distribution and the values of r_u correspond quite well with Equation (2.13) and Figure 2-9. The reattachment point lies 5-7 times the step height downstream of the step. The area near and slightly downstream of the reattachment point, appears to be the critical area of attack on a bed protection, see also Chapter 3. It is clear that 10 times the step height downstream of the step, the flow is still far from uniform.

Figure 2-13e shows the deviations from hydrostatic pressure ($\rho g(h-z)$) made dimensionless with the pressure accompanying the velocity on top of the step ($\Delta p/2\rho u_1^2$). Near the reattachment point, where the flow hits the bottom, the pressure is high. In the recirculation zone the pressure is low due to the entrainment of water into the mainstream. This pressure pattern causes flow from the reattachment point in a direction opposite to that of the mainstream.

For calculations of stone stability and scour it is important to have an idea of the degree of recovery of the flow as a function of the distance downstream of the step. Figure 2-13f is derived from Hoffmans, 1993, and shows the transition of the turbulence in the mixing layer to that of the new wall boundary layer. The value of k/u_1^2 downstream of the step is 0.045 as found from Figure 2-9, Section 2.4, while the final turbulence in the boundary layer is calculated according to an expression similar to Equation (2.11).

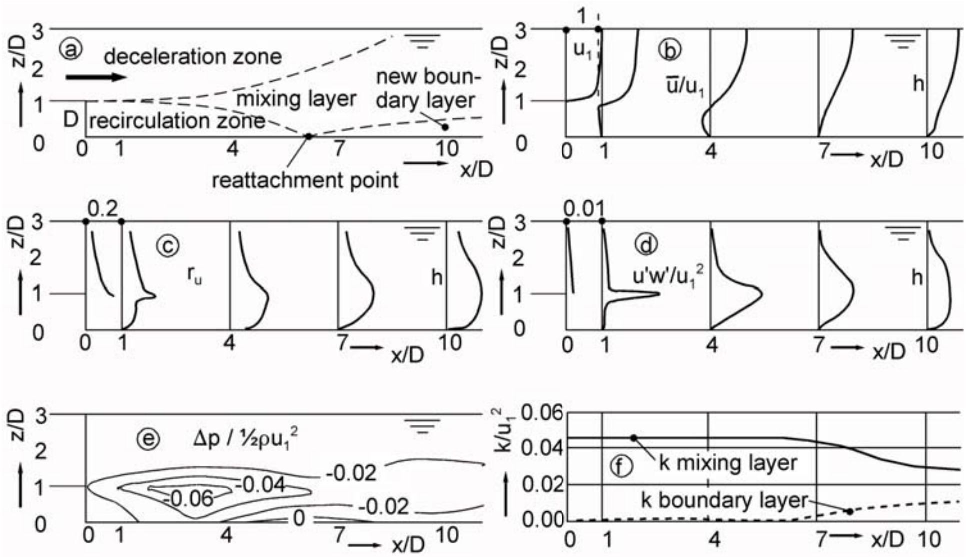

Figure 2-13 Flow phenomena in backward-facing step

For locations downstream of the reattachment point, Hoffmans proposes to calculate the relative depth averaged turbulence, r as follows:

$$r(x) = \sqrt{0.5\,k_0 \left[1 - \frac{D}{h}\right]^{-2} \left[\frac{x}{\lambda} + 1\right]^{-1.08} + 1.45\,\frac{g}{C^2}} \tag{2.15}$$

in which λ is a relaxation length ($\lambda \approx 6.67\ h$). k_0 is the relative turbulent energy in the mixing layer ($\approx$ 0.045). The factor 0.5 accounts for the substitution of k_0 by the depth-averaged r. The velocity on top of the step is substituted by the velocity downstream with a factor ($1-D/h$), derived from the continuity equation. x is the *distance downstream of the reattachment point* ($\approx 6D$ downstream of the step). The second term represents the equilibrium value of r in uniform wall flow.

2.5.3 Vertical constriction and expansion (sill)

A sill is a good example of a vertical constriction (Blom, 1993). Computations were done and experiments were carried out in a flume. Figure 2-14 shows measurements obtained with the experiments. The slopes of the sill were such that no real flow separation occurred.

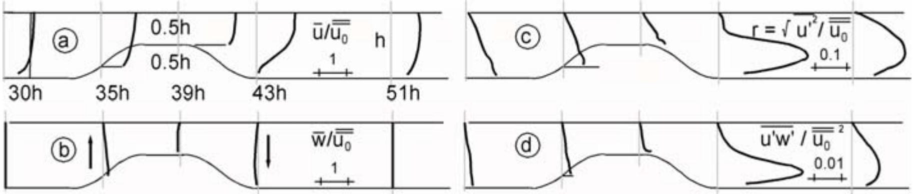

Figure 2-14 Flow characteristics around sill (from Blom, 1993)

In Figure 2-14a we see the changes of the velocity: a logarithmic profile before the sill, accelerating flow and a more rectangular profile on top, decelerating flow and almost separating flow beyond the sill and a slow recovery to a logarithmic profile in uniform flow again, see also Figure 2-7. The vertical velocities are negligible, except near the sill (Figure 2-14b).

The turbulent fluctuations r_u, related to u_0 (the average velocity upstream, Q/A, Figure 2-14c), are rather small before the sill and remain more or less the same on the sill, leading to a lower local r-value (the average velocity doubles). Beyond the sill the turbulence increases considerably, due to the deceleration and the energy transfer to turbulence (this internal process is the so-called Carnot energy-loss in basic hydraulics). The r-value beyond the sill is about 0.3. This is slightly lower than in a mixing layer, where it would be about 0.35 - 0.4 (~ 0.2 related to the velocity on top of the sill, twice as much related to the average downstream velocity). So, even without flow separation, this flow pattern closely resembles the pattern of a backward facing step and deceleration always causes a considerable increase in turbulence.

In the acceleration zone, the shear stress tends to increase, which is visible in Figure 2-14d. In the deceleration zone, the shear stress is great with the maximum velocity gradient and much smaller at the bottom. It is good to bear in mind that the shear stress behind a constriction does not determine the load; the Reynolds normal stresses can be more important. Beyond the sill, the flow slowly recovers to uniform flow.

2.5.4 Horizontal expansion

The situation is similar to the backward facing step of Section 2.5.2, but now the flow is constricted in a horizontal direction, see e.g. the research by Tukker, 1997 and Zuurveld, 1998; Figure 2-15 shows some results.

A mixing layer originates beyond the sudden expansion, but the growth is now limited compared with the situation in Section 2.4.1 due to bed friction (Figure 2-15a). Both near the bottom and the surface, the velocity spreads into the stagnant zone, again with recirculation beyond the expansion (Figure 2-15b). The turbulent fluctuations show large differences between the bottom and the surface (Figure 2-15c): the effect of the mixing layer is only visible at the surface, at the bottom the friction dominates. Figure 2-15d shows the peak velocities near the bottom. These peak velocities ($\bar{u} + 3\sigma = [1+3r] \times \bar{u}$) are often held responsible for stability and erosion. It appears that the highest peak velocities near the bottom occur in the mainstream and not in the mixing layer.

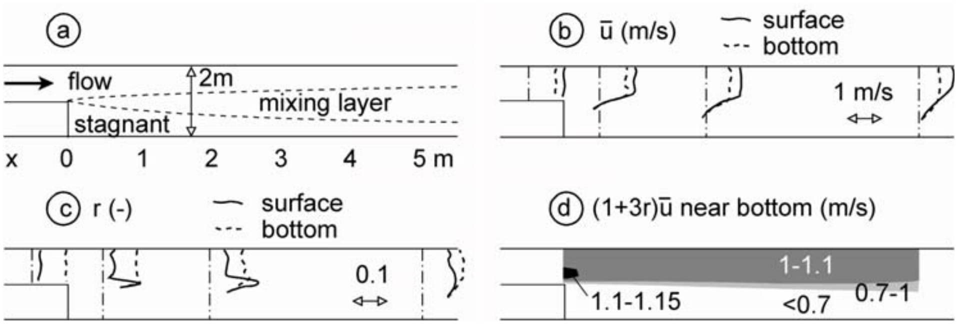

Figure 2-15 Flow characteristics in horizontal expansion (Zuurveld, 1998)

2.5.5 Horizontal constriction and expansion (groyne)

Abutments and groynes are clear examples of horizontal constrictions. Natural variations in the width and protruding points in a river show the same phenomena. For a study on scour and grain stability, 1/6 of the width of a flume was blocked and the flow pattern for this situation was investigated, see Ariëns,1993.

Figure 2-16 shows measured values of the velocity (vertically averaged and averaged over the turbulence period, see Equation (2.1)), $\bar{u}$, the relative turbulence (related to the *local* value of $\bar{u}$), r and the (absolute) value of the peak velocity, $(1+3r)\ \bar{u}$. The acceleration and deceleration are clearly visible in the figure showing the average velocity. Acceleration again causes a decrease in the *relative* turbulence from 8% to less than 5%, with again an increase in the deceleration zone. In the mixing layer, the relative turbulence reaches high values. (The mixing layer is now curved, because the flow is forced from the side of the flume to the centre.)

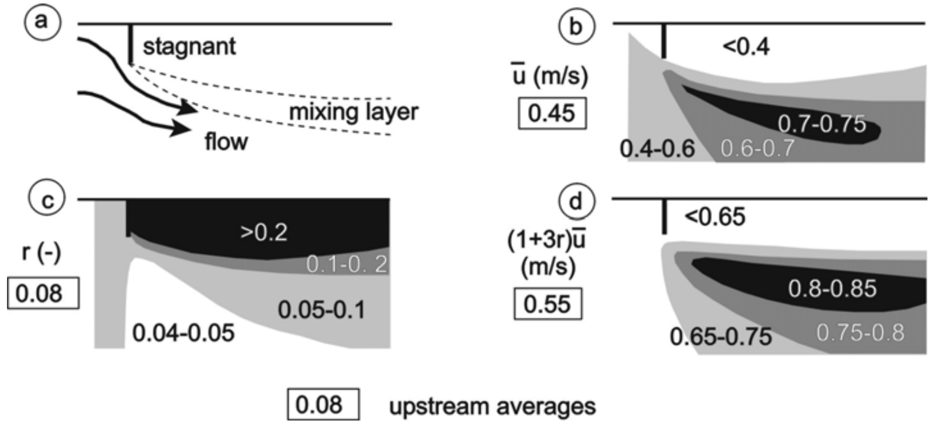

Figure 2-16 Flow characteristics for a horizontal constriction (from Ariëns, 1993)

The diagram representing peak velocities contains a large zone with high values. Now, in contrast with the expansion in Section 2.5.4, the area with the highest peak velocities coincides largely with the mixing layer. This is, however, due to the velocity pattern associated with the acceleration around the groyne.

2.5.6 Detached bodies

Examples of detached bodies are bridge piers or jetty piers, nautical structures and offshore platforms. An elementary shape of a detached body, which has been studied intensively, is a cylinder. As this shape is important to the engineering practice, it is useful to pay attention to the complicated flow characteristics around a cylinder. To a large extent, the flow pattern and the forces in the flow are determined by flow separation around the cylinder but as the separation depends on the roughness of the surface and the *Re*-number (now defined as uD/ν where D is the diameter of the cylinder), it is still very hard to predict. Moreover, the flow around a cylinder is truly 3-dimensional.

Potential flow (see e.g. Le Méhauté, 1976) is a classical approach to flow around a cylinder. It neglects some of the above-mentioned complications but is still useful as a starter. Figure 2-17a and c show streamlines and velocities (relative to the upstream average) according to potential flow. Figure 2-17 b and d show measurements near the bottom. A comparison of a with b shows the merits and the limitations of the potential approach. In the upstream area and in the acceleration area, roughly up to the centre-line of the cylinder, the similarity is quite good. In the deceleration area, the flow separates and the whole pattern changes. In the wake of the cylinder, a recirculation occurs like we have seen before in the case of an expansion. The potential theory predicts a doubling of the velocity alongside the cylinder. The measurements near the bottom show the same, albeit with lower absolute values near the bottom.

Figure 2-17e shows the water surface according to potential flow and as measured. Along the central streamline, the water in the vicinity of the cylinder decelerates in front of the body, accelerates at the sides and decelerates again at the back. The associated water level, rises, falls and rises again (see points 1-2-3) with the local velocity head ($u^2/2g$). The conversion of potential energy into kinetic energy on its way from 1 to 2 can be considered a "ride downhill", from 2 to 3 it is "uphill" into an area with increasing pressure. In a perfect fluid there is no friction and the pressures in 1 and 3 are equal (pressure recovery), hence the net force on the body is zero.
The viscosity of a real fluid, however, changes the picture completely. The friction in the boundary layer stops the uphill motion between 2 and 3 causing flow separation from the body. The flow separation and wake result in an extra drag force on the body (in addition to the friction along the body).

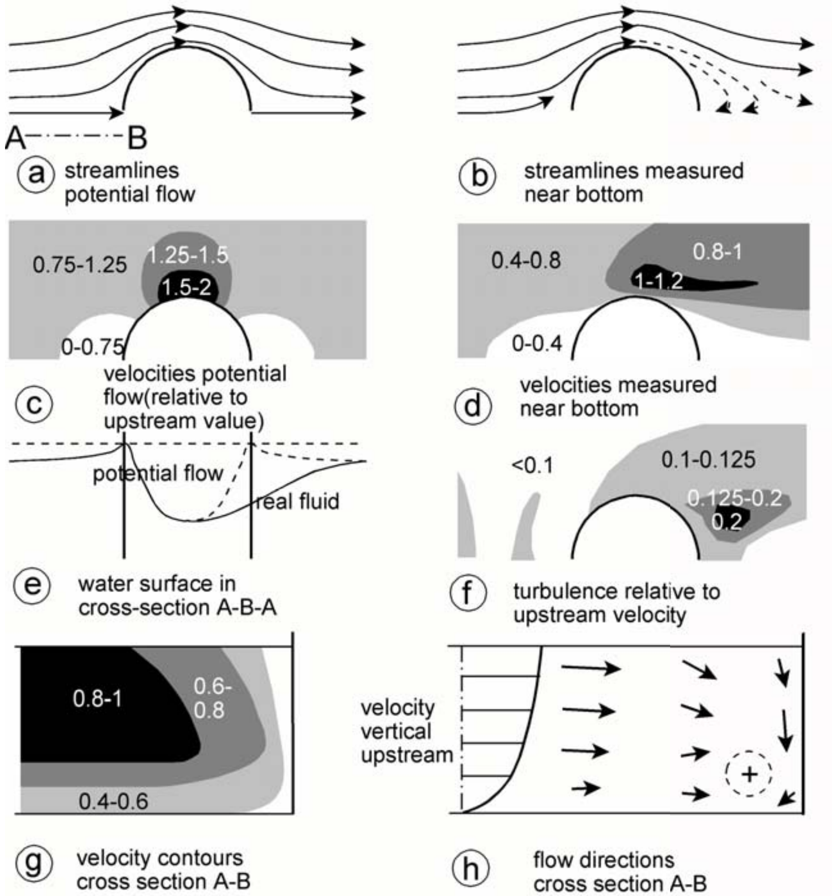

Figure 2-17 Flow details around cylinder (from Melville and Raudkivi, 1977)

Figure 2-17 illustrates some more interesting details of the flow around a cylinder (from Melville and Raudkivi, 1977). Figure 2-17f shows the turbulence intensity ($r = \text{rms}\{u'\}/u_0$) near the bottom. In the acceleration area on the sides of the cylinder the turbulence intensity diminishes, as can be seen in the diagram. In the mixing layer between the wake behind the cylinder and the main flow, the maximum intensity, around 0.2, is found.

Figure 2-17g and h show the vertical cross-section at the centre-line of the cylinder. Figure 2-17g gives the contour lines of the velocity in the direction of the main flow. The deceleration is clearly visible. The rise in pressure that comes with this decrease of velocity causes another interesting phenomenon. The pressure rises with $\Delta p = 1/2\rho\, u^2$. The velocity near the water surface is higher than near the bottom, hence Δp, with a constant piezometric level across the waterdepth, results in a pressure gradient downwards. This causes a vertical jet (Figure 2-17h) which plays an important role in the scour around a cylinder, see Chapter 4. Theoretically, the vertical velocity in the jet can reach the same value as the approach velocity but measurements show less than half that value. At the foot of the pier, a circulation is visible. In Chapter 4 we will see that this is the beginning of the so-called horse-shoe vortex. Some authors claim that this vortex is always present around a pier, others say that it originates in the scour hole in front of the pier and that without scour there is no vortex. More information on flow around cylindrical piers is found in e.g. Melville and Raudkivi, 1977, Raudkivi and Etttema, 1985, Zdravkovich, 1997 and Ahmed and Rajaratnam, 1998.

2.6 Load reduction

Sometimes it can be desirable to reduce the flow as a means of protection or to avoid the need for heavy protections. Figure 2-18 shows three ways to reduce the flow, two at a river bank and one at an outflow.

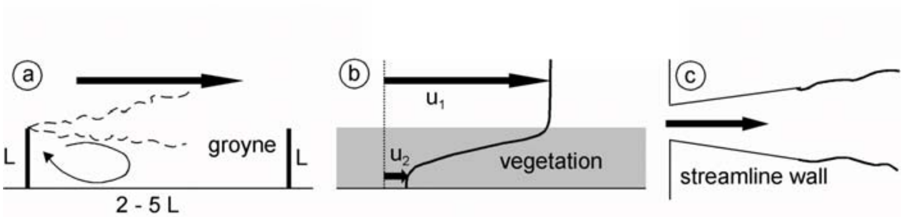

Figure 2-18 Possible reduction of flow induced loads

Groynes keep the main flow at a distance from the banks. In rivers, they are usually part of a complete river training scheme, not just to reduce the velocity, but also to maintain a stable channel with sufficient depth for navigation. In this case the focus is on flow reduction. At the tip of a groyne, the flow separates and an eddy is formed (see Figure 2-18a). To keep the main flow away from the bank, the distance between two groynes should be less than 5 times their length. A more strict demand is the eddy pattern. For a proper guidance of the main flow, there should be one eddy between two groynes which requires a distance between groynes of about twice the groyne length. The velocity in the eddy is about 1/3 of the velocity in the main current, so the reduction is about 2/3. For more information see Jansen, 1979. Groynes, however, are very costly and cause their own problems, e.g. scouring holes at the tip. The cost-benefit relation should be considered in all cases.

Instead of using groynes, flow near the bank can be reduced in a diffuse way by increasing the roughness. This can be done by making the bed more rough or by placing resistance elements in the flow. The latter is the case when vegetation, e.g. reed or trees, is planted along the bank. It is possible to express the resistance of a roughened bed with a Chezy-coefficient. A first approximation of the velocity reduction is found by assuming that the water level slope is equal for the main flow and the bank, see Figure 2-18b:

$$u = C\sqrt{h\,I} \;\rightarrow\; \frac{u_2}{u_1} = \frac{C_2}{C_1}\sqrt{\frac{h_2}{h_1}} \tag{2.16}$$

In Chapter 12 examples of roughness values for various types of vegetation will be given.

When the turbulence caused by a mixing layer has to be avoided or postponed till a point where the velocity is lower has been reached, walls are often streamlined with slopes of 1 : 8 or even less to prevent flow separation, see Figure 2-18c. This is often done with outflows at culverts and similar structures. The fact that the flow does not separate within the streamlined walls does not mean that there is no increase in the turbulence intensity. Deceleration always causes more turbulence, so, the main advantage is the reduction of the mean flow velocity.

2.7 Summary

To design a protection, or to judge whether a protection is necessary, detailed information on the velocity field is needed to determine the loads. This can be obtained from measurements, a flow model or a sketch, based on overall discharge data and insight into the flow phenomena. In fact, every preliminary design should start with a simple sketch on a sheet of paper.

The same is true for turbulence in the velocity field, albeit that the determination of turbulence is much more difficult and uncertain. The turbulent velocity fluctuations are a result of velocity differences, either between a flow and a wall (wall turbulence) or between two fluid bodies (free turbulence). Turbulence is always coupled with loss of kinetic energy, so, especially in deceleration areas great turbulence can be expected. In accelerating flow the relative turbulence is less, but the shear stress increases. The turbulent fluctuations cause so-called Reynolds stresses: a normal stress due to the velocity fluctuations in one direction and a shear stress due to the correlation between the fluctuations in two directions.

For uniform (wall) flow, relatively simple, partly empirical relations are available for the average velocity (like $\bar{u} = C\sqrt{RI}$) and accompanying expressions for wall roughness and turbulence which originates at the wall. The boundary layer in uniform flow covers the whole waterdepth. Mixing layers play an essential part in free turbulence and occur in many situations in hydraulic engineering. Another example of free turbulence can be found in jet flow, which is also frequently encountered in flow situations near hydraulic structures. A typical value for wall turbulence is r (= $rms\{u'\}/\bar{u}$) ≈ 0.1 and for free turbulence r_{max} ≈ 0.2-0.3. These values also depend on where $\bar{u}$ is defined.

Flow situations around bodies, like (bridge) piers, through vertical constrictions, like sills and through horizontal constrictions, like abutments and groynes, all show acceleration followed by deceleration with the accompanying turbulence. The acceleration leads to higher velocities than in uniform flow situations, causing a greater load on any interface in the neighbourhood. The same is true in the deceleration zone where the increase of the Reynolds stresses is caused by an increase of the turbulent fluctuations. Section 2.5 gives several examples of combinations of turbulent wall flow and free flow. You are advised to study these thoroughly and, above all, to imagine what is happening. This will be all the more fruitful when coupled with observations of flow phenomena everywhere around you in rivers, ditches or in your sink.

Finally, some examples of load reduction are presented in Section 2.6.

2.8 APPENDICES

2.8.1 Basic equations

The basic equations for fluid motion (conservation of momentum and mass) will be used for demonstration purposes only and mathematical purity is sacrificed in search of simplicity. For more detail, see e.g. Schlichting (1968) or Le Méhauté (1976).

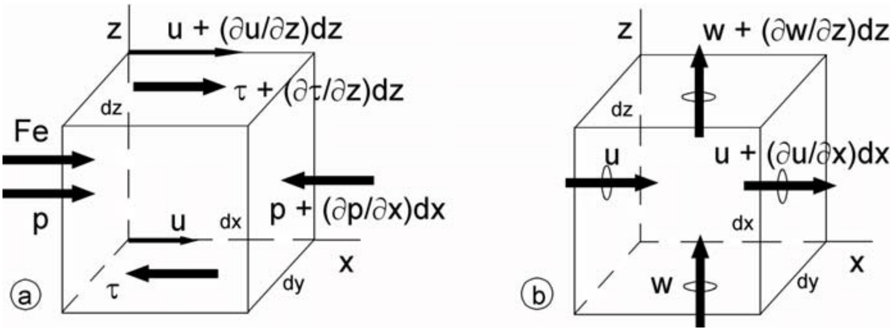

Figure 2-19 Forces and flow with regard to dxdydz

The momentum equation is in fact Newton's second law: $F = ma$. For a 2-dimensional flow the resulting force in the x-direction is (see Figure 2-19a):

$$F_x = -\frac{\partial p}{\partial x} dx\,(dy\,dz) + \frac{\partial \tau}{\partial z} dz\,(dx\,dy) + Fe(x) \tag{2.17}$$

The second term on the right-hand side of Equation (2.17) represents the rate of change of viscous shear stress perpendicular to the flow (in the z-direction there is a shear stress of equal magnitude). The viscous shear is an internal stress caused by the transfer of molecular momentum. For a Newtonian fluid: $\tau = \mu\, \partial u/\partial z$, so, $\partial\tau/\partial z = \mu($

$\partial^2 u/\partial z^2$), in which μ is the dynamic viscosity. The external force, Fe, can result from gravity, which is supposed to work in z-direction, so, in the x-direction, external forces are neglected. The acceleration in x-direction can be written as:

$$a_x = \frac{Du}{Dt} = \frac{\partial u}{\partial t} + \frac{\partial u}{\partial x}\frac{dx}{dt} + \frac{\partial u}{\partial z}\frac{dz}{dt} = \frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + w\frac{\partial u}{\partial z} \quad (\frac{dx}{dt} = u,\ \frac{dz}{dt} = w) \tag{2.18}$$

With $m = \rho\,(dxdydz)$ this leads to:

$$\begin{aligned} m\,a_x = \rho\,(dx\,dy\,dz)\left(\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + w\frac{\partial u}{\partial z}\right) = \\ F_x = -\frac{\partial p}{\partial x}dx\,(dy\,dz) + \mu\frac{\partial^2 u}{\partial z^2}dz\,(dx\,dy) \end{aligned} \tag{2.19}$$

Dividing by ($dx\ dy\ dz$) finally gives the (simplified) *Navier-Stokes equation* for the x-direction, which is valid for both laminar and turbulent flow:

$$\rho\left(\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + w\frac{\partial u}{\partial z}\right) = -\frac{\partial p}{\partial x} + \mu\frac{\partial^2 u}{\partial z^2} \tag{2.20}$$

local inertia, convective inertia, pressure gradient, viscous shear

The *continuity equation* (conservation of mass) for a non-compressible fluid with constant density states that the net flow through a fluid element without a free surface must be zero (see Figure 2-19b):

$$\frac{\partial u}{\partial x} + \frac{\partial w}{\partial z} = 0 \tag{2.21}$$

For turbulent flow, the values of velocity and pressure in the equations can be split up. For the velocity this gives: $u = \bar{u} + u'$, in which $\bar{u}$ indicates the average value and u' the turbulent fluctuation. In order to work with average values, Equation (2.20) can be averaged over the turbulence period (see Figure 2-2). The linear terms, like $\partial u/\partial t$ become:

$$\begin{aligned} \overline{\frac{\partial u}{\partial t}} = \frac{1}{T}\int_0^T \frac{\partial u}{\partial t}dt = \frac{\partial}{\partial t}\frac{1}{T}\int_0^T \left(\bar{u} + u'\right)dt = \\ \frac{\partial}{\partial t}\frac{1}{T}\int_0^T \bar{u}\,dt + \frac{\partial}{\partial t}\frac{1}{T}\int_0^T u'\,dt = \frac{\partial \bar{u}}{\partial t} + 0 = \frac{\partial \bar{u}}{\partial t} \end{aligned} \tag{2.22}$$

since:

$$\bar{u} = \frac{1}{T}\int_0^T u\,dt \quad \text{and} \quad \bar{u}' = \frac{1}{T}\int_0^T u'\,dt = 0 \tag{2.23}$$

The same holds for w and p, the linear terms for laminar and turbulent flow are expressed in the same type of function. This is not true, however, for the quadratic terms:

$$\overline{u^2} = \frac{1}{T}\int_0^T u^2\;dt = \frac{1}{T}\int_0^T \left(\bar{u}^2 + 2\cdot\bar{u}\cdot u' + u'^2\right)\,dt = \bar{u}^2 + \overline{u'^2}$$

$$\text{since}: \; \frac{1}{T}\int_0^T 2\cdot\bar{u}\cdot u'\;dt = 2\,\bar{u}\;\frac{1}{T}\int_0^T u'\;dt = 2\cdot\bar{u}\cdot 0 = 0 \tag{2.24}$$

$$\text{and}: \; \frac{1}{T}\int_0^T u'^2\;dt = \overline{u'^2} \; \text{(mean value of } u'^2 \neq 0\text{ !)}$$

Similarly:

$$u\,w = (\bar{u} + u')\cdot(\bar{w} + w') = \bar{u}\,\bar{w} + u'\bar{w} + w'\bar{u} + u'w' \quad \text{hence:} \quad \overline{uw} = \bar{u}\,\bar{w} + \overline{u'w'} \tag{2.25}$$

and Equation (2.20) becomes (note: the viscous term is of the second order but linear):

$$\rho\left(\frac{\partial\bar{u}}{\partial t} + \bar{u}\frac{\partial\bar{u}}{\partial x} + \bar{w}\frac{\partial\bar{u}}{\partial z} + \overline{u'\frac{\partial u'}{\partial x}} + \overline{w'\frac{\partial u'}{\partial z}}\right) = -\frac{\partial\bar{p}}{\partial x} + \mu\frac{\partial^2\bar{u}}{\partial z^2} \tag{2.26}$$

local inertia — conv.inertia by mean vel. — conv.inertia by fluct.vel. — press. grad. — visc. shear

With the continuity equation, this can be rewritten to obtain the so-called *Reynolds equation*:

$$\rho\left(\frac{\partial\bar{u}}{\partial t} + \bar{u}\frac{\partial\bar{u}}{\partial x} + \bar{w}\frac{\partial\bar{u}}{\partial z}\right) = -\frac{\partial\bar{p}}{\partial x} + \mu\frac{\partial^2\bar{u}}{\partial z^2} - \rho\left(\frac{\partial\overline{u'^2}}{\partial x} + \frac{\partial\overline{u'w'}}{\partial z}\right) \tag{2.27}$$

inertia — press. — visc. — Reynolds-stresses

|----------------------------| |------------

mean values — turb. fluct.

The Reynolds-stresses are a consequence of working with time-averaged values. The turbulent shear stress ($u'w'$) is usually much larger than the viscous shear stress, while the turbulent normal stress (u'^2) can become important in situations with flow separation, see the examples in Section 2.5.

2.8.2 Why turbulence?

Everybody who has been on an aeroplane in rough weather, or on a bike in gusty wind, has some idea of turbulence. And everybody who looks carefully at the fluid motion in a river, or even in a straight flume in a laboratory, will notice the turbulent character of the flow. Turbulent means the opposite of laminar. Laminar flow is smooth and orderly, while turbulent flow is chaotic and fluctuates, even when the flow as a whole looks quiet.

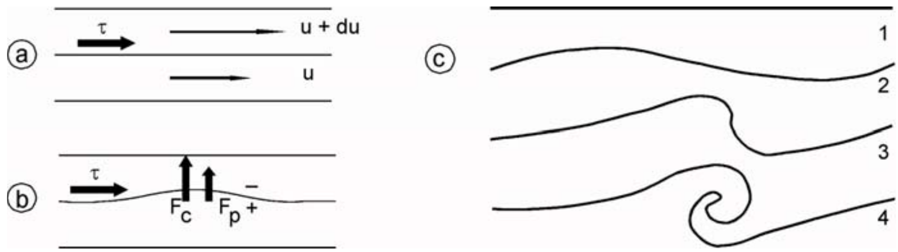

Figure 2-20 (In)stability of laminar flow

Figure 2-20 (from Le Méhauté, 1976) shows the origin of turbulence. Between two fluid layers with different velocities, e.g. the outflow in Figure 2-1b, one always finds friction. A small disturbance of the flow, regardless of its cause, will induce forces that increase the disturbance. The curve in the flow line results in a centrifugal force (F_c in Figure 2-20b). The same curve causes a decrease of the velocity in the lower layer and an increase in the upper layer (continuity!) resulting in a pressure difference (F_p in Figure 2-20b), which follows from the Bernoulli equation (piezometric head + velocity head = constant). The pressure difference and the centrifugal force work in the same direction and tend to increase the instability. The result is an increasing undulation, the so-called Kelvin-Helmholtz instability, see Figure 2-20c, which shows successive stages of this instability. In the end, packets of fluid travel in every direction: the turbulent eddies.

The centrifugal force ($= \rho\, u^2/R$) and the pressure force (proportional to the difference in velocity, hence to $\rho\, u(\partial u/\partial x)$, see Section 2.8.1) are destabilizing forces. They are related to the non-linear convective inertia terms in the basic equations and are expressed in the same units. The viscous damping (represented by $\mu(\partial^2 u/\partial z^2)$, see also Section 2.8.1), has a stabilizing effect as an increase in the path length in the undulation causes an increase in viscous friction. All forces can be expressed with a characteristic *velocity difference*, U, over a characteristic length, L. The ratio between destabilizing and stabilizing forces then leads to the well-known Reynolds-number:

$$Re = \frac{\rho \cdot U^2 / L}{\mu \cdot U / L^2} = \frac{U \cdot L}{\upsilon} \tag{2.28}$$

where μ is the so-called dynamic viscosity and υ the kinematic viscosity ($\upsilon = \mu / \rho$). υ is a property of the fluid and is independent of geometry or flow velocity. So, there will always be a value for U and L, for which the flow will be naturally unstable.

The chosen value L in the Re-number, depends on the geometry of the flow. In river flow the depth is a logical choice; for the flow around a cylinder, the diameter is a clear choice. For the velocity difference U, the velocity itself is the most appropriate value (the difference between the flow and the river bed or the stagnant zone behind the cylinder). Typical values for the transition between laminar and turbulent flow are Re = 1000-2000. For normal circumstances, with $\upsilon \approx 10^{-6}$ m^2/s, this means that the flow is already turbulent in a flume with a depth of 0.1 m, and a velocity of a few centimetres per second! In the hydraulic engineering practice, flow will always be turbulent.

From the foregoing we have learned that turbulence is caused by a velocity gradient perpendicular to the main flow direction. This gradient can exist between a wall and the flow or between two flow zones. In the first case we speak of *wall turbulence* and in the second case of *free turbulence*, see Figure 2-21 a and b. Wall turbulence exists where water flows along a bottom, a bank or a body. Free turbulence occurs where two masses of water are forced to move along each other with different velocities, like the outflow of a jet in a stagnant ambient fluid or the wake behind a body, in a hydraulic jump or a breaking wave.

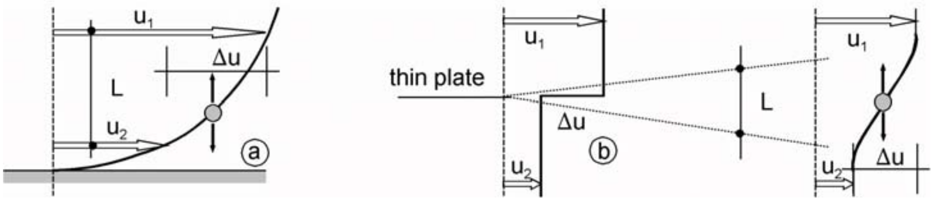

Figure 2-21 Wall turbulence and free turbulence

The energy cascade

We have seen that turbulence is induced by a velocity difference, that is too large for the molecular viscosity which can otherwise prevent the formation of turbulent eddies. Another way to look at turbulence is as follows. A rubber ball, falling from some height, gains kinetic energy due to the loss of potential energy. This kinetic energy is transferred into heat, by means of friction between the ball and the ground and internal friction inside the ball due to deformation, until it finally comes to a halt on the ground. Usually, a ball has to bounce several times before the energy transfer is completed. Something similar goes on in decelerating flow. The kinetic energy of the mean motion is transferred into heat by the viscous shear stress: $\tau = \rho\ \upsilon(\partial u/\partial z)$. υ has a fixed value, coupled with the physical properties of the fluid. Maximum values of $\partial \bar{u}/\partial z$ in the flowing water are too small to transfer all of the kinetic energy into heat at once. Turbulence is a means of increasing the viscosity (hence the so-called turbulent viscosity) and can be interpreted as the process that allows the kinetic energy of the main flow to dissipate, leaving the system as heat due to viscous friction.

Another analogy is dissolving sugar in a cup of tea. The molecular diffusion (the mechanism that makes the sugar dissolve) and the gradient of the sugar concentration in a cup at rest have limited values and it can take some time before all of the sugar is dissolved. Stirring is an effective way of increasing the surface with a gradient such that diffusion can take place. Turbulence can then be seen as the pre-processing in the treatment of "waste" kinetic energy.

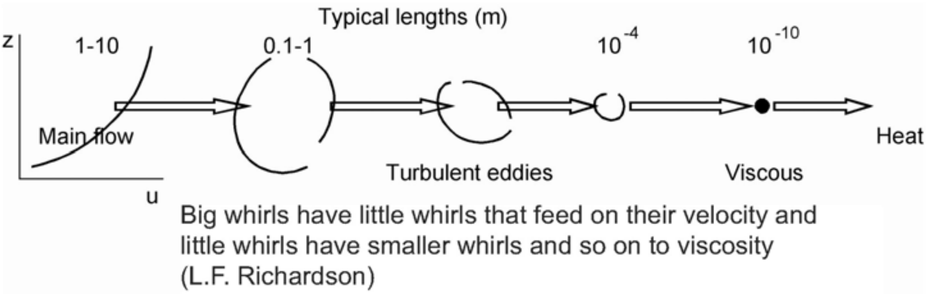

Figure 2-22 Energy cascade in turbulent motion

Figure 2-22 shows the transfer process with some typical lengths. Only in the smallest eddies, the velocity gradient is large enough for the viscous friction to play an effective role in the transfer into heat.

Illustration: Energy dissipation in hydraulic jump

To demonstrate some of the phenomena discussed in the preceding, the flow in a hydraulic jump will be reviewed in detail. A hydraulic jump can be seen as the ultimate energy dissipator. From the one-dimensional momentum equation one can derive:

$$\frac{h_2}{h_1} = \frac{1}{2}\left(\sqrt{1 + 8\,Fr_1^2} - 1\right) \quad (Fr_1 = u_1 / \sqrt{gh_1}) \tag{2.29}$$

Figure 2-23a shows the profile and the main characteristics as defined in basic hydraulics. Since we are interested in what is happening inside the flow, we take a look "under the hood" of the hydraulic jump. Figure 2-23b to f show the results of measurements in a hydraulic jump with a Fr-value of 4, as performed by Rouse, 1958.

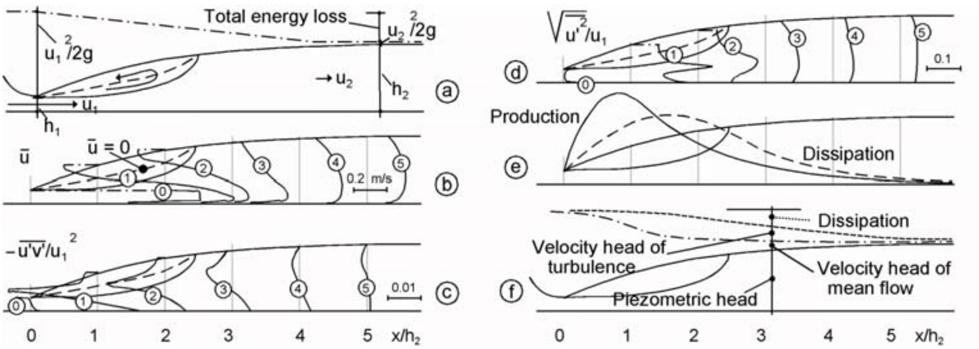

Figure 2-23 Flow and turbulence in hydraulic jump (Fr = 4, from Rouse, 1958)

Figure 2-23b shows the velocity in the jump, $\bar{u}$, averaged over the turbulence period. The spread of the velocity is clearly visible, with a reversal of the flow in the roller. Figure 2-23c shows the turbulent shear stress. Where the gradient of $\bar{u}$ is maximal, the largest values of $u'w'$ can be found. Immediately after the roller starts, the turbulent shear stress starts to grow quickly. The turbulent fluctuations in the flow direction are shown in Figure 2-23d. The fluctuations also grow where the velocity gradient is large, but they are then diffused through the water and the location of the maximum turbulence shifts. Turbulent fluctuations in the vertical direction are slightly smaller than those in the flow direction, but they show the same pattern. Figure 2-23e gives the production and dissipation of turbulence, while Figure 2-23f shows the energy content of the flow. The figures show that turbulence is indeed an intermediate phase in the transformation of kinetic energy of the main flow into heat by means of viscous shear stress. Velocity gradients are important in the transfer of energy from the main motion to the turbulent eddies.

Finally, since all motion ends in heat, it is interesting to compute the raise in temperature in a hydraulic jump. For a jump with $Fr = 4$, $h_1 = 1$ m and $u_1 = 12.5$ m/s, given that 4000 J is needed to create a temperature rise of 1 °C in 1 kg water, the total raise in temperature is less than 0.01 °C!! So, turbulence may cause a lot of excitement in engineering circles, it leaves the water cold.