Impact of the Delta Works on the Recent Developments in Hydraulic Engineering

K.W. Pilarczyk*

* Rijkswaterstaat, Hydraulic Engineering Institute, Delft, The Netherlands

k.w.pilarczyk@dww.rws.minvenw.nl; k.pilarczyk@ planet.nl

Disaster in 1953 was a turning point in the Dutch policy on flood protection. The Delta Works, which followed this disaster, contributed significantly to the recent worldwide developments in hydraulic engineering. A brief overview is presented of some important items related to closure techniques, erosion, scour and protection, specifically.

I. Introduction

Figure 1. The Netherlands with and without flood protection

The Netherlands is situated on the delta of three of Europe's main rivers: the Rhine, the Meuse and the Scheldt. As a result of this, the country has been able to develop into an important, densely populated nation. But living in the Netherlands is not without risks. Large parts of the Netherlands are below mean sea level and water levels, which may occur on the rivers Rhine and Meuse (Fig. 1). High water levels due to storm surges on the North Sea or due to high discharges of the rivers are a serious threat for the low-lying part of the Netherlands. A total of about 3000 kilometres of primary flood protection structures protect areas, which are vital for the existence of Dutch nation. Flood protection measures have to provide sufficient safety to the large number of inhabitants and the ever increasing investments. Construction, management and maintenance of flood protection structures are essential conditions for the population and further development of the country.

History shows that flooding disasters nearly always resulted into actions to improve the situation by raising dikes or improving the discharge capacity of the rivers. The disastrous flood of 1953 marks the start of a national reinforcement of the flood protection structures. The recent river floods of 1993 and 1995 did accelerate the final stages of this reinforcement programme. History also shows that neglect is the overture for the next flooding disaster. In an attempt to improve on this historic experience the safety of the flood protection structures in the Netherlands will be assessed regularly. Maintaining the strength of the dikes at level according to the legally prescribed safety standards is the main goal of this safety assessment.

II. History

To understand the historical development of the protection by dikes in the Netherlands, it is essential to know the aspect of the gradual land subsidence in combination with rise of the sea level with respect to the land and also the decreasing deposits of soil by the North Sea and the rivers (Fig. 2) [1].

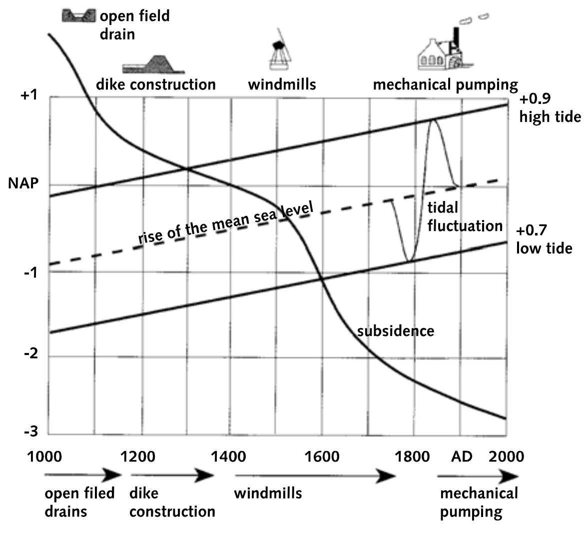

Figure 2 Land subsidence & sea-level rise in the past 1000 years

Mechanisation and industrialisation led to improve drainage. Making use of these techniques even land under water could be recovered (i.e., some lakes and Zuyder Sea). At the same time, however, the lowering of the ground level accelerated by which the effects of possible flooding only increased. Strengthening flood defences

was seen in the past as the means of effectively addressing this threat. In this way, from the middle of the thirteenth century until today in total about 550,000 ha of land was reclaimed. However, during the centuries much of the previously reclaimed land was lost by attack of the sea mainly due to storm surges which many times caused destruction of the dikes. Another phenomenon was the occurrence of landslides along tidal channels (most dikes were situated on loose soils), thus causing disappearance of dikes. During high storm surges the sea also eroded this land. Nevertheless every time there was the spirit of the people to push back the sea. Most of the lost land was reclaimed again, despite the ever-occurring storm surges.
However, the continuous land subsidence and increased sea level rise have serious implications for the safety of the land protected by dikes, dunes and other defence structures along the coast and the lower parts of the main rivers. Sea level rise increases the risks of overtopping and the ultimate collapse of these structures during storms.

The history of the Netherlands is marked by storm surge disasters. The most recent disaster took place on the first day of February 1953 when a north westerly storm struck the south western part of the Netherlands (Delta area). The storm surge level reached 3 to 3.5 m above normal high water and exceeded design storm surge levels about 0.5 m at some places. Some dikes could not withstand these levels, so that at several hundreds of places the dikes were damaged and/or broken, over a total length of 190 km. Through nearly 90 breaches 150,000 ha of polder land inundated. This caused the death of 1835 people and 100,000 persons had to be evacuated; moreover a lot of live-stock drowned and thousands of buildings were damaged or destroyed. This disaster gave a new impulse to improve the whole sea defence system in the Netherlands. The resulting Delta Plan included the strengthening of existing dikes and shortening the length of protection in the Delta area by closing-off estuaries and a tidal river. Together with the Delta works the low-lying 'polder land' of the Netherlands has reached already a relative high degree of safety against storm surges.

III. Delta Works

To indicate the role and importance of Delta Works in the proper context it is necessary to keep in mind the vulnerability of the Netherlands and the Dutch history concerning the battle against the sea. As it was mention before, over half of the Netherlands lies below sea level. Just how vulnerable the country is to flooding was demonstrated by storm-surge on the night of 1 February 1953. The results were catastrophic due to many breaches of dikes. Flooding caused by storm surges were nothing new to the Netherlands, but this time the nation was stunned by the extent of a disaster unparalleled for centuries. It was The Netherlands' worst disaster for 300 years. The hardest hit areas were the province of Zeeland, the southern part of South Holland and the western part of North Brabant. The bewilderment and shock felt by people in the rest of the Netherlands when they learnt of the extent of the flooding soon gave way to determination, and great efforts were made to reseal the breached dykes. The last breach, near Ouwerkerk on Schouwen-Duiveland, was resealed at the beginning of November 1953.

Rarely have the people of the Netherlands been as united as when they decided that such a catastrophe should never happen again. We may say that Disaster 1953 was a turning point in Dutch approach to flood protection and flood management. The outcome of this determination was the Delta Project.

A. *The Delta Project*

On February the 21st, 1953, the Delta Commission was founded, directed by the director-general of the Department of Waterways and Public Works. Its aim was to draw up a plan to ensure two goals would be reached:
1. Drain the areas that flood regularly during high water levels and protect them from the water,
2. Protect the land from getting brackish.

The body of measures proposed by this committee forms the Delta Plan. The aim of the Plan is enhance safety by radically reducing the length of and reinforcing the coastline. These measures are laid down in 1958 in the Delta Act.

The Delta Plan Project's principal goal was to improve the safety of the southwest Netherlands by considerably shortening and reinforcing the coastline. It was decided that dams should be constructed across inlets and estuaries, considerably reducing the possibility of the sea surging into the land once more. Freshwater lakes would form behind them. Roads along the dams would improve access to the islands of Zeeland and South Holland.

Dams could not be constructed across the New Waterway or the Western Scheldt, as these important shipping routes to the seaports of Rotterdam and Antwerp had to be kept open. The safety of these areas was to be guaranteed by substantially reinforcing the dykes.

The Delta Project is one of the largest hydraulic engineering projects that has ever been carried out anywhere in the world. New hydraulic engineering techniques were gradually developed for the construction of the eleven dams and barriers of various size which we re built over a period of thirty years.

In the early 1970s the realization grew that it was important to preserve as much of the natural environment as possible and this point of view has left its mark on the Delta Project. As a result the original plans were changed (see Section B).

The Delta Project was likely carried out in a random order. Work began on the relatively simpler parts, so that the experience gained could be used during the construction of larger, more difficult dams across inlets and estuaries with strong tidal currents. That was how the Delta works progressed: new hydraulic engineering techniques were first applied on a small scale and then used in the larger, more complicated, projects. In this way, as much as possible was able to be learnt during the construction process. Also, when determining the order of execution of projects, the materials and manpower available were taken into account. On the basis of these considerations, it was decided to carry out the works in the following order (date’s of completion) (Fig. 3):

Figure 3. The Deltaplan

1958: Storm surge barrier in the Hollandse IJssel
1960: Zandkreek Dam
1961: Veerse Dam
1965: Grevelingen Dam
1970: Volkerak Dam with lock complex
1971: Haringvliet Dam with discharge sluices
1972: Brouwers Dam
1983: Markiezaatskade
1985: Eastern-Scheldt storm surge barrier
1987: Oesterdam
1987: Philipsdam and locks
1997: Flood-control Oude Maas and New Waterway (Rotterdam).

B. *The Eastern Scheldt (Oosterschelde)*

While the Haringvliet dam and the Brouwers dam were nearing completion (1971), preparations had already begun for the construction of the dam across the mouth of the Eastern Scheldt, the last, largest and also most complex part of the Delta Project. Three islands were constructed: Roggenplaat, Neeltje Jans and Noordland. A pumped sand dam was built between the latter two. In the remaining channels the first steel towers were built for the cableway, as it was planned to dam the Eastern Scheldt using this well-tried method. Its completion date was set for 1978.

At the end of the 1960s however protests were voiced about the project. Scientists became aware of the special significance of the flora and fauna in and around the Eastern Scheldt. The Sandbars and mud flats exposed at low tide are important feeding grounds for birds, and the estuary is a nursery for fish from the North Sea. Fishermen and action groups made sure that the scientific findings were heard by the government and parliament. A heated debate flared up. Opponents of the dam believed that the safety of the region could be guaranteed by raising the height of the dykes along the Eastern Scheldt. The inlet would then remain open and saline. The equally vigorous supporters of the solid dam, for example agricultural and water boards, appealed to the emotions of the Zeelanders, asking whether the consequences of the flood disaster of 1953 had already been forgotten.

C. *The Storm Surge Barrier*

A compromise was reached in 1976: a storm surge barrier, which would stay open under normal conditions but which could be closed at very high tides. The construction of the storm surge barrier meant a break with the policy that the Public Works Department and the hydraulic engineering contractors in the Netherlands had pursued in working from small to large and from relatively simple to complex. The storm surge barrier needed expertise that had yet to be developed and experience that had vet to be gained. Extensive research was carried out to determine the feasibility of building the storm surge barrier, taking full account of the interests of the environment, flood protection, and the fishing and shipping industries.

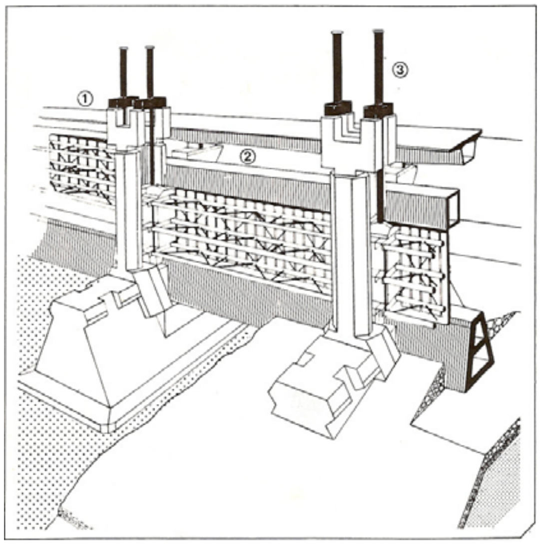

Figure 4. Storm-surge Barrier Eastern Scheldt (view of structural components)

The actual construction of the storm surge barrier also had to be thoroughly studied. The solution was a barrier consisting of pre-fabricated concrete and steel components that were assembled in the three channels at the mouth of the Eastern Scheldt. 65 colossal concrete piers form the barrier's backbone. A stone sill and a concrete sill beam were placed between each of the piers, and the openings could be closed with steel gates. Concrete box girders were placed on top of the piers to form a road deck.

The seabed also needed special consideration. A new technique was required to prevent the strong current in the mouth of the river from washing away the sand on which the piers were to stand. The solution was to place the piers on mattresses filled with graded layers of sand

and gravel which would allow water to flow through but trap the sand.

The construction of the storm surge barrier also required the development of special equipment. The 'Mytilus' made its appearance in the estuary to compact the seabed, followed by the 'Jan Heijmans' which laid asphalt and dumped stones, the 'Cardium' to position the mattresses, the 'Ostrea' to lift, transport and position the piers and the mooring and cleaning pontoon 'Macoma'. These are very special ships designed for just one purpose: to construct the storm surge barrier. New measuring instruments and computer programs were also developed, so that engineers working 30 to 40 metres below the surface could position components with such precision that the maximum error would be just a few centimetres.

The Cardium laid the first mattress in November 1982 and the Ostrea placed the first pier in August 1983. Work progressed quickly. There were virtually no technical setbacks; only the cost turned out to be higher than expected. The storm surge barrier was 30% more expensive than estimated. On 4 October 1986 Her Majesty Queen Beatrix officially opened the storm-surge barrier. The Eastern Scheldt has remained open and flood protection has been achieved. On average the barrier has to be closed once a year because of storms.

D. Eastern Scheldt Project

On The Eastern Scheldt Project comprises more than the construction of a storm-surge barrier in the mouth of the estuary. In the eastern part the Markiezaatskade and Oesterdam, and the Philipsdam have been constructed along with shipping locks to create a tide-free navigation way between Rotterdam and Antwerp. The Zoom meer forms a fresh water lake behind these dams. To prevent the salty Eastern Scheldt water from mixing with the fresh Zoommeer water, the locks are fitted with a special fresh-salt water separation system. From the Zoommeer a discharge channel carries fresh water into the Western Scheldt. The Eastern Scheldt Project also involved far-reaching adaptations of the South-Beveland canal.

The storm-surge barrier in the Eastern Scheldt has been commissioned by Rijkswaterstaat (Public Works Department), and was built by the Oosterschelde Stormvloedkering Bouwcombinatie (Dosbouw/Ostem), a joint venture by a number of contractors.

The detailed description of Eastern-Scheldt Barrier can be found in “Design Plan Oosterschelde” (1994) [2]. The total design documentation consists of five books. This book is a translation of Book 1: Total design and design philosophy. The other books are only available in Dutch. More general findings obtained from Delta Works are summarized in “Closure of Tidal Basins” (1984, 1987) [3]. Both books are backed up by the experiences of a great number of engineers and other professionals involved in this large project. In the course of the project, a wealth of knowledge and experience has been acquired. These books have been written to make this knowledge available for future use.

E. Significance of the Deltaworks

Besides shortening the total length of the dikes by 700 kilometres, the Delta works had many other advantages. Firstly, the agricultural freshwater supply was improved. Because the border between freshwater and saltwater was moved further west, less freshwater was required to balance the freshwater-saltwater division. The excess water could be transported to the north of the Netherlands, in the direction of the Ijsselmeer (Ijssel lake), where extra freshwater was welcomed to improve the water conditions. Secondly, the complete water balance of the Delta area was improved. Thanks to the construction of the major and auxiliary dams, the streams in this area were able to be manipulated more easily. Different types of sluices made it possible to allow fresh water in, or polluted or excess water out. Thirdly, the construction of the Delta works encouraged traffic between the many islands and peninsulas. Large parts of the province of Zeeland had literally been isolated for centuries. The building of the Zeeland Bridge together with a tunnel under the Westerschelde tunnel (2003) also helped increase mobility. Fourthly, the inland waterways’ shipping was supported by the Delta works. In 1976, Belgium and the Netherlands signed a contract that would regulate the shipping between the ports of Antwerp and Rotterdam. Obviously, this agreement had to be taken into account when building the Delta works. It was realized by construction of compartment dams and navigation locks. Lastly, the Delta works have influenced new developments in the areas of nature and recreation. Understandably, a number of nature reserves were irreparably damaged, but as compensation, new nature reserves have emerged at different sites. Nowadays, dry shores are sometimes used as recreational areas. Whether or not nature has benefited from the Delta works will remain an unsolved debate. However, there is no doubt over the need for durable water management, in which safety, prosperity, and nature are taken into account.

Other developments: In addition to the construction of new dams and barriers, at several places, existing dams had to be heightened. This was especially the fact in the western parts of the islands (Walcheren, Schouwen, Goerree) and along the waterway of Rotterdam and the Western Scheldt. The dikes needed reinforcement because they were not directly protected by the large works. It is a common misconception that the Delta works were only built to replace dikes. In most of the cases, building a delta work was much quicker, and cheaper than reinforcing existing dikes. Since the building and strengthening of dikes are time consuming and expensive, another delta work was built to the west of Rotterdam at the end of the 20th century. The movable barrier, called the 'Maeslant Barrier', can close off the New Waterway when water levels are threatening the dikes in the environment.

Due to the recent climate change and the rise in sea level, high water levels are more likely to occur near the coasts of Zeeland and Holland. The number of people that live in the polders, several metres below sea level, has actually increased since the flood of 1953. The general consensus among scientists is that the reinforcement of dikes and the construction of dams and barriers are in no way the final siege in the battle against the sea.

IV. Contribution Delta Works to Developments in Hydraulic Engineering

The Netherlands has long and varied experience of hydraulic engineering, and particularly of constructing dykes, digging canals, draining polders and building locks, bridges, tunnels and ports. That experience is also put to use in the oft-shore industries - in the construction of production platforms, but also in foreign projects in South Korea, Bangladesh, Bahrain and others, for example. Working in and with water has given the Dutch a world-wide reputation, and the Zuyder Zee project, which not only protected large areas of the country from flooding but also provided about 160,000 ha. of new land, and the Delta project, which is also to protect the Netherlands from the ravages of the sea, are outstanding examples of their expertise in this field. Impact of the Delta Works is manifested in a number of transitions in hydraulic engineering approaches, for example, from deterministic into probabilistic approach, from traditional to innovative techniques and materials, etc.. Some elements of these developments and expertise will be briefly highlighted below.

A. Design Methodology and Innovative Execution

The Delta Project was a challenge to Dutch hydraulic engineers. It was evident that past experience and existing techniques would not be sufficient to enable dams to be constructed across the wide and deep tidal channels. The tidal range in the Delta is approximately three metres and the water flows in and out twice a day with powerful currents shifting enormous quantities of sand. Weather conditions in the estuaries were often unfavourable and North Sea storms produced powerful waves. New techniques had to be developed quickly so that the Delta Project could be carried out.

The towering caissons were adapted and improved. Man-made fibres (geosynthetics) were used for the first time to protect the seabed and to clad the dykes. The traditional seabed protection method, which involved covering the seabed with large mats made of willow wood and weighted with stone, was gradually replaced.

Changes took place step by step. It was decided to implement the Delta Project by working from small to large so that technological progress would keep pace with the growth in experience.

Prefabrication became a common technique and in addition to new materials, new equipment was also very valuable. Sluice caissons were developed. A cableway with gondolas was developed to tip stone into the channels.

Hydrodynamic study techniques were refined by developing laboratory tests. The computer gradually made its entrance. Measuring techniques and weather forecasts became more accurate. The Delta Project took about 25 years to complete. A new age was dawning for hydraulic engineering, also in international context.

Design, management and maintenance of hydraulic structures and flood protection systems are of great importance. The probabilistic design technique is suitable for this task. This technique gives a clear view of the weak points of a structure and the various ways in which it can be optimized. Because of complexity of Eastern Scheldt project the probabilistic approach has been applied on a large scale.

Probabilistic approach: Probabilistic methods were introduced in the design of the storm surge barrier mainly for two reasons [4]. After the Disaster 1953 the Delta Committee stipulated that primary sea-retaining structures had to provide full protection against storm surge levels with an excess frequency of 2.5×10^{-4} time per year. In case of conventional defences, such as dikes, an extreme water level (combined with a maximum extrapolated single wave) may be used as a design criterion, because overtopping is considered to be the most important threat to dikes. However, this approach was/is unsuitable for a storm surge barrier. The structure consists of various components (concrete piers, steel gates, a sill, a bed protection and a foundation), which have to be designed on different load combinations providing most dangerous threat to the structural stability. The probability density function of the load was derived by integrating the multidimensional probability density function of wave spectra, storm surge levels and basin levels using the transfer function of the structure. To ensure consistent safety throughout the structure, probabilistic analyses taking into account the stochastic character of the loads and the structural resistance are performed for the main components. To assess the safety of the barrier as a sea defence system, a risk analysis was performed using the fault tree technique. More information hereabout can be found in [4].

Application of probabilistic approach in the design of storm-surge barrier Eastern Scheldt was a starting point of introducing this technique in other fields of hydraulic and coastal engineering in the Netherlands and elsewhere. Probabilistic approach and risk analysis are actually standard items included in civil engineering education.

Failure modes: The probabilistic approach gives the probability of failure of a (elements of a) structure and takes into account the stochastic character of the input variables. This is in contrast to a deterministic design method which is based on fixed values, for example, mean or extreme values. Studies have been performed using a failure mechanism based on the stability and displacement of structural elements and protection materials.

A focal point in the feasibility study was whether or not the subsoil would be able to deliver the necessary counteracting forces against storm surge. Together with specialists from several countries, the Delft Soil Mechanics Laboratory completed a series of laboratory tests to assess subsoil characteristics.

In addition, a full-scale test was conducted outdoors to study the behaviour of a caisson subjected to cyc1ic loading. New computation techniques were developed to predict the behaviour of the subsoil under design conditions.

Delft Hydraulics Laboratory was given the task of predicting design loads on the barrier due to tides, storm surges and waves. Model experiments were performed in the wind-wave facilities of the laboratory using small scale models of the proposed structures, which were subjected to random seas.

Verification of design: Not all hydraulic or coastal structures or their components are understood completely; moreover, the existing design techniques represent only a certain schematisation of reality. Therefore for a number of structures and/or applications the verification of design by more sophisticated techniques can still be needed.

Delta Works were an excellent example where large scale verification was applied. A large number of prototype tests were executed for verification of small scale results and validation some design assumptions. Examples of that were tests on caissons, prototype scour tests, tests of bank protection systems, etc., but also analyses of failures from prototype.

Whilst certain aspects, particularly in the hydraulic field, can be relatively accurately predicted, the effect of the subsequent forces on the structure (including transfer functions into sublayers and subsoil) cannot be represented with confidence in a mathematical form for all possible configurations and systems. Essentially this means that the designer must make provisions for perceived failure mechanisms either by empirical rules or past experience. However, using this approach it is likely that the design will be conservative. In general, coastal structures (i.e. dikes, revetments, sea walls, etc.) are extended linear structures representing a high level of investment. The financial constraints on a project can be so severe that they may restrict the factors of safety arising from an empirical design. It is therefore essential from both the aspects of economy and structural integrity that the overall design of a structure should be subject to verification. Verification can take several forms: physical modelling, full-scale prototype testing, lessons from past failures, etc.

Engineers are continually required to demonstrate value for money. Verification of a design is often expensive. However, taken as a percentage of the total costs, the cost is in fact very small and can lead to considerable long-term savings in view of the uncertainties that exist in the design of hydraulic and coastal structures. The client should therefore always be informed about the limitations of the design process and the need for verification in order to achieve the optimum design.

B. Closure Techniques. Sand Closures

A large number of closure techniques were applied in the scope of Delta Works (Fig. 5). More information hereabout can be found in "Closure Tidal Basins" [4] and in [5]. Only sand closures will be discussed more in detail.

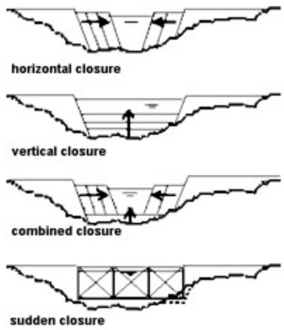

Figure 5. Schematization of various closure techniques

Sand closures

One of the latest developments of the closure-techniques for tidal gaps is the closing with sand as the only building material. The sand closure is carried out by supplying sand either by dumping from the boat or by a pipeline that takes a water-sediment suspension to the head of the dam. The suspension runs off over the fill, where the sand deposits due to decelerating water velocities and will built out the dam in that way. Due to narrowing of the closure gap, which causes increasing velocities, and due to the fact that a part of the sediment supplied by the pipe will reach the closure gap in suspension a loss of sand will appear. The material (sand) should have either sufficient weight (diameter) to resist erosion or be supplied in such large quantities so that the main portion is not carried away by the current.

In 60's a number of tidal channels in the Netherlands, and some in Germany and Venezuela, were closed successfully by pumping sand in the gap. The experience was restricted to the gaps with the maximum current velocities in the final closure-stage not higher, than 3m/s. Until 1982 in all cases the dredger (=pumping) capacities that were needed were estimated by the semi-empirical method as described in [3]. The new developments with larger gaps, higher velocities, and new calculation and execution techniques are described in [6].

Assumptions and definitions: Before calculations of the dredger capacity and sand loss start some assumptions have to be made on the dimension of the closure dam (Fig. 6).

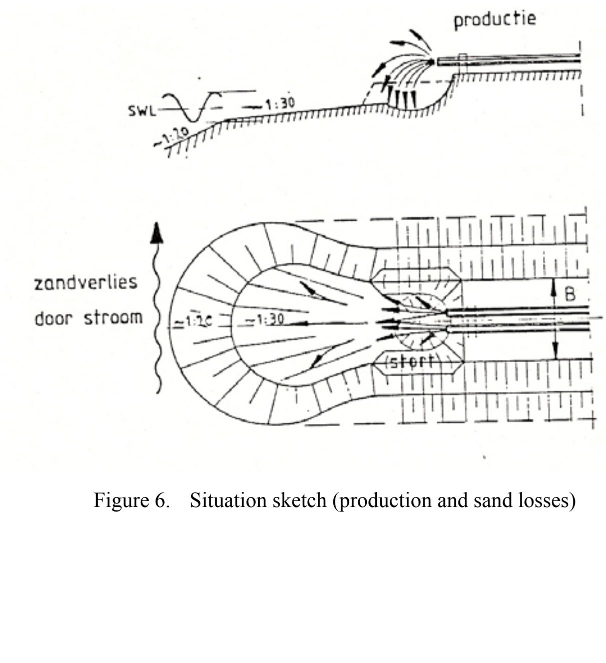

Figure 6. Situation sketch (production and sand losses)

The crest-elevation of the closure dam (temporary function') depends on astronomical tide and wind set-up (usually a recurrence interval of 5-10 years is taken) and the run-up. The crest width depends on the number of pipes and the working space needed for bulldozers on the fill. The pipes will be used in pairs: one for production while the other can be lengthened by connecting a new pipe section (B ~ 20m for one pair of pipes). The slopes of the dam in the tidal range will vary from 1:30 to 1:50, depending on the wave action. The slopes under low-water will become in the order of 1:10 to 1:20. The side-slopes of the dam are mostly created by bulldozers and can be taken equal to 1:5. In order to reduce the sand loss in the final stage of the closure, the remaining gap should be situated in a shallow part of the channel. This is due to the fact that less sand should be brought in to make the same progress as in the case of a deeper gap.

A distinction is made between gross and net loss of sand. The gross loss is the sand" that deposits outside of the closure dam profile but inside the final dam profile and is of importance for the time involved in the closure operation. The net loss is the sand that is taken beyond the profile of the final dam and will determine the actual loss. However, in the standard calculation method no distinguish is made between these two kinds of loss and only the total loss is calculated (Fig. 7).

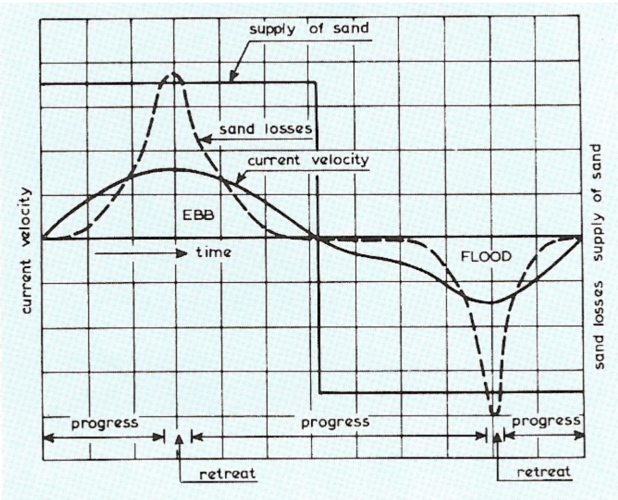

Figure 7. Sand losses as a function of time and tidal motion

Figure 8 Execution of sand closure

The duration (and cost) of the operation depend further on the location and the available diameter of sand where the sand for the dam is dredged. Travel distance should be kept as short as possible. The increasing grain size will diminish the losses and thus will influence the success of the operation in a positive way. Success of the operation also depends on the working method, the lay-out of the pipelines, the sand-water ratio of the spoil and the effective working time of the dredgers. In the final gap bulldozers and draglines operate on the fill to control the sand deposition above the water level (Fig. 8). However, this very important influence of the working method on the total sand-losses is not involved in the calculating procedure. This positive effect can be treated as an extra safety in the calculating results especially in the light of many limitations of this method. The detailed overview of past sand closures and design information can be found in Sand Closures [6].

C. Scour and Bottom Protection

Scour is a natural phenomenon caused by the flow of water in rivers and streams. Scour occurs naturally as part of the morphological changes of rivers and as result of structures man-made. Scour prediction and bed protection during closure operations are essential aspects during design and execution (Fig. 9).

In case of Eastern Scheldt Barrier, to protect the seabed from erosion caused by the increased speed of the currents, a 500-600 metre wide area on either side of the barrier had to be covered. Had this not been done the channels might have eroded to such an extent that the barrier would be endangered.

The protection consists of concrete weighted erosion mats, aprons of stone filled asphalt and stone weighted mastic asphalt slabs. Highly advanced techniques were developed especially for this operation. The concrete weighted erosion mats, manufactured in the factory at the Sophia harbour later used to make the block mattresses, were laid by special vessel. The asphalting operations were carried out by the Jan Heijmans, which has later been converted into a gravel/stone dumper to be used for filling in the space between the foundation filter mattresses.

Scour and Flow slides: Generally the scour process can be split up into different time phases. In the beginning the development of scour is very fast, and eventually an equilibrium situation will be reached. During closing operations of dams in estuaries or rivers (Fig. 9), which take place within a limited time, equilibrium will not be attained. Not only during the construction, but also in the equilibrium phase it is vital to know both the development of scour as function of time and whether the foundation of the hydraulic structure is undermined progressively.

Experience has shown that due to shear failures or flow slides or small scale shear failures at the end of the bed protection, the scour process can progressively damage the bed protection, leading eventually to the failure of the hydraulic structure for which the bed protection was meant.

In the scope of the Dutch Delta works, a systematic investigation of time scale for two and three-dimensional local scour in loose sediments was conducted by Delft Hydraulics and 'Rijkswaterstaat' (Department of Transport and Public Works). From model experiments on different scale and bed materials, relations were derived between time-scale and scales for velocity, flow depth and material density [7,8]. In addition empirical relations were found in order to predict the steepness of the upstream scour slope [9].

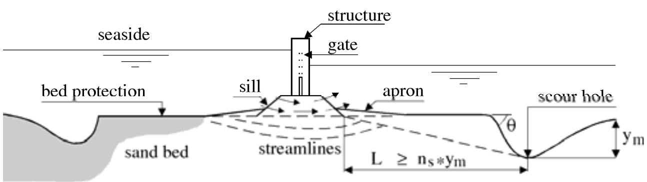

Figure 9. Schematization of a storm surge barrier

Besides the systematic investigation, design criteria for the length of the bed protection were deduced which were based on many hundred of shear failures and flow slides occurred along the coastline of estuaries in the south-western part of the Netherlands [3].

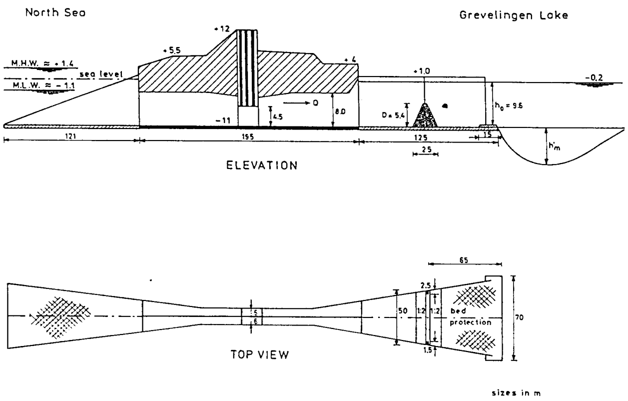

Figure 10 Sluice Brouwersdam

Verification with prototype data: Within the scope of research activities with respect to scour behind the storm surge barrier and compartment dams in the Eastern Scheldt, some field experiments were carried out [9]. For this purpose the sluice in the Brouwersdam was chosen, which was built to refresh the brackish water in the Grevelingen Lake for environmental reasons (Fig. 10). A 5.4 m high sill was constructed at the lake side of the sluice with two side constrictions equal to 2.5 m on the left side and 1.5 m on the right side. The flow depth was about 10 m and the length of the bed protection from the toe of the sill measured about 60 m. The effective roughness of the bed protection is estimated to be 0.4 m.

The experiments were executed to study the influence of clay layers to scour and to verify scour relations obtained from scale models [9,10,11]. The agreement was very satisfied except the stability of the upstream scour surface. In general, the prediction of upstream slope of scour hole is still a weak point and needs further investigation. The Dutch efforts into knowledge of scour were completed in the scope of PhD-thesis by G. Hoffmans [10], and finally compiled in Scour Manual [12].

D. Stability of Cover Layers

A large number of various sloping structures were realized in the scope of the Delta Works. Such structures as abutments of barriers, jetties, compartment dams, navigation channels and some dikes needed a proper slope protection. However, the inventory studies at the end of 70-s indicated a lack of proper design criteria for revetments. Therefore a number of basic studies were initiated for this purpose. The results of these studies were later transformed/translated into more general design standards, which often became the international standards.

Rubble structures and riprap:

All the older design were mostly based on (simplified) formula of Hudson dated from 50’s, which gained its popularity due to its simplicity and the status of US Army Corps. However, the problems with using this formula started in 70’s with introduction of random waves and the necessity of transformation of regular waves into irregular waves. In 80’s, the number of testing facilities and test results with random waves became so large that the necessity of new design formulas became evident. The new research in 80’s provided more understanding of failure mechanisms and new more sophisticated formulae on stability and rocking of rubble mound structures and artificial armour units.

Formulae developed by Van der Meer [13] by fitting to model test data, with some later modifications, became standard design formulations. However, the reason of this development was quite different than above mentioned. To explain this we have turn back to 70’th when the author was involved in Delta project, the largest project of damming tidal gaps in the Netherlands, following the necessary actions after flood disaster in 1953. The author has discovered at that time that there was little known on stability of cover layers under wave attack, and that the existing formulations (Hudson, Irribaren, Hedar) were not perfect. He was strengthened in his suspicion by research of John Ahrens, a brilliant researcher from US Corps of Engineers, who was probably too far ahead in time for a general acceptance. Even in his home organisation he never gained the recognition, as he deserved; his work was never mention in US Shore Protection Manual. Ahrens [14] performed extensive tests on riprap stability and the influence of the wave period; the test were conducted in the CERC large wave tank (with regular waves). Pilarczyk [15] continued to replot Ahrens’s data and obtained surprising similarity with the later design graph by Van der Meer (1988). This work by Ahrens and his later research on dynamic stability of reefs and revetments, together with work by van Hijum on gravel beaches [16], were the reason for the author to prepare a proposals for a systematic research on static and dynamic stability of granular materials (rock and gravel) under wave attack. This program, commissioned early 80's to the Delft Hydraulics, was successfully realized under direct guidance by Van der Meer in 1988 [13].

The basic structure of the Van der Meer formula is such that the stability number H/ΔD is expressed in terms of natural or structural boundary conditions, for example (the sample formula is valid for rock under plunging waves):

$$\frac{Hs}{\Delta D_{n50}} = 6.2P^{0.18}\left(\frac{S}{\sqrt{N}}\right)^{0.2}\frac{1}{\sqrt{\xi_m}} \qquad (1)$$

in which: H_s = significant wave height, Δ = relative mass density, D_{n50} = nominal stone diameter, P = permeability coefficient representing composition of the structure, S = damage level, and ξ_m = surf similarity parameter (Iribarren number).

The work of Van der Meer is now generally applied by designers and it has considerably reduced (but not eliminated) the need to perform model experiments during design process. We have always to remember that each formula represents only a certain schematisation of reality. Moreover, as far as these formulas are based on experiments and not based on fully physical understanding and mathematical formulations of processes involved, each geometrical change in the design may lead to deviation in the design results, and to the need of performance of model investigation. Another advantage of the Van der Meer formulae over the formula of Hudson is the fact that the statistical reliability of the expression is given, which enables the designer to make a probabilistic analysis of the behaviour of the design.

Following the same philosophy, Van der Meer and others have modified and extended the formulae for the stability of rock to many other aspects of breakwater design such as stability of some artificial units, toe stability, overtopping and wave transmission. However, these latest formulae (overtopping and transmission) are still in very rudimentary stage and need further improvement and extension. It concerns especially such structures as submerged reefs with a wide crest where all design aspects (stability, transmission, and functional layout) are not understood properly yet. Some experience with these structures is obtained in Japan, however, the generally valid design criteria are still absent.

What has been said for slopes under wave attack is largely valid for slopes and horizontal bottom protection under currents. The designer has a number of black box design tools available, but the understanding of the contents of the black box is far from complete. Specifically when these black box design formulae are used in expert systems, one may in the end be confronted with serious mistakes. If an experienced designer still realises the shortcomings and limitations of the black box formula, the inexperienced user of the expert system can easily overlook the implication of it.

Although a reliable set of design formulae is available [17,18], the main challenge in the field of rubble mound structures is to establish a conceptual model that clarifies the physical background of it. This will require careful experimental work, measuring the hydrodynamic conditions in the vicinity of the slope and inside the breakwater/structure [19].

Block revetments:

It should be stressed that the proposed developments for breakwaters were partly stimulated/initiated by early developments in understanding and quantification of physical processes in block revetments [20,21,22].

Wave attack on revetments will lead to a complex flow over and through the revetment structure (filter and cover layer), which are quantified in analytical and numerical models [23,24]**.** The stability of revetments with a granular and/or geotextile filter (pitched stones/blocks, block mats and concrete mattresses) is highly influenced by the permeability of the entire revetment system. The high uplift pressures, induced by wave action, can only be relieved through the joints or filter points in the revetment (Figure 11). The permeability of the revetment system is a decisive factor determining its stability, especially under wave attack, and also it has an important influence on the stability of the subsoil. The permeability of a layer of closely placed concrete blocks on a filter layer with and without a geotextile has been investigated in recent years in the Netherlands in the scope of the research programme on stability of revetments

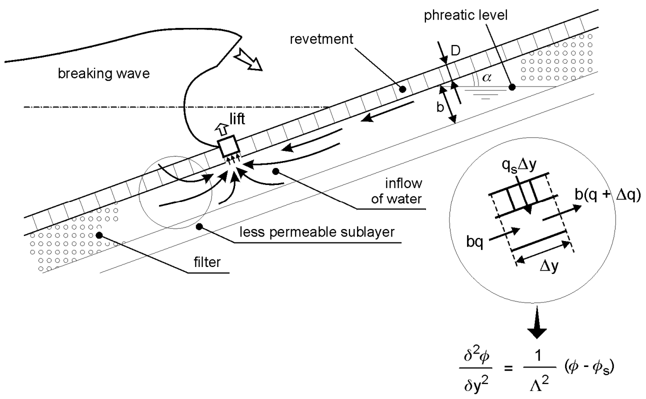

Figure 11. Physical processes in revetment structure

The usual requirement that the permeability of the cover layer should be larger than that of the under layers cannot usually be met in the case of a closed block revetment and other systems with low permeable cover layer (i.e. concrete geomattresses). The low permeable cover layer introduces uplift pressures during wave attack. In this case the permeability ratio of the cover layer and the filter, represented in the leakage length, is found to be the most important structural parameter, determining the uplift pressure. The schematised situation can be quantified on the basis of the Laplace equation for linear flow (Figure 11). In the analytical model nearly all-physical parameters that are relevant to the stability have been incorporated in the "leakage length" factor. For systems on a filter layer, the leakage length Λ is given as:

$$\Lambda = \sqrt{\frac{bDk}{k'}} \tag{2}$$

where: Λ = leakage length (m), D = thickness of the revetment cover layer (m), b = thickness of the filter layer (m), k = permeability of the filter layer or subsoil (m/s), and k' = permeability of the top (cover) layer (m/s).

The pressure head difference, which develops on the cover layer, is larger with a large leakage length than with a small leakage length. This is mainly due to the relationship k/k' in the leakage length formula. The effect of the leakage length on the dimensions of the critical wave for semi-permeable revetments is apparent from the following equation:

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} \tag{3}$$

where: H_{scr} = significant wave height at which blocks will be lifted out (m); $\xi_{op} = \tan\alpha/\sqrt{(H_s/(1.56T_p^2))}$ = breaker parameter (-); T_p = wave period (s); Δ = relative mass density of cover layer = $(\rho_s - \rho)/\rho$, and f = stability coefficient mainly dependent on structure type and with minor influence of Δ, tanα and friction.

Filters

The research on revetments has proved that the stability of a revetment is dependent on the composition and permeability of the whole system of the cover layer. Formulas have been derived to determine the permeability of a cover layer and filters, including a geotextile. Also, stability criteria for granular and geotextile filters were developed based on the load – strength principle, allowing application of geometrically open filters, and thus allowing optimisation of composition and permeability of revetments. It is obvious that only a certain force exceeding a critical value can initiate the movement of a certain grain in a structure. That also means that applying geometrically closed rules for filters often may lead to unnecessary conservatism in the design and/or limitation in optimisation freedom (see Figure 11). Also, it often results in execution problems especially when strict closed filter (with many layers) has to be executed under water under unstable weather conditions [1,25].

In the scope of these studies, also the internal strength of subsoil has been studied in terms of critical hydraulic gradients. It was recognised that to reduce the acting gradients below the critical ones a certain thickness of the total revetment is needed. This has resulted in additional design criteria on the required total thickness of revetment to avoid the instability of the subsoil. That also means that granular filter cannot always be replaced by geotextile only. For high wave attack (usually, wave height larger than 0.5m or high turbulence of flow) the geotextile functioning as a filter must be often accompanied by a certain thickness of the granular cushion layer for damping hydraulic gradients. All these design criteria can be found in [24,26].

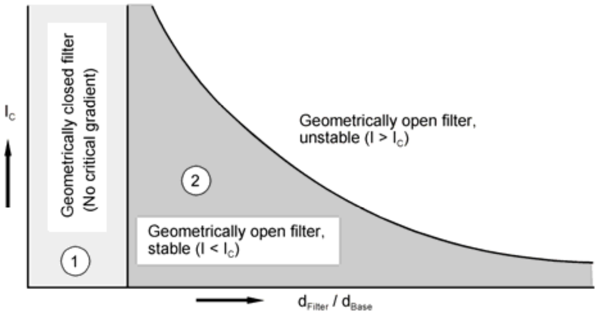

Figure 12. Application of granular filters

The main problem in extension of these achievements to other applications (other revetments, filter structures, bottom protection, breakwaters, etc.) is the lack of calculation methods on internal loads (i.e., hydraulic gradients) for different structural geometry and composition. Also the geometrically open filter criteria need further development. Research in these fields is still needed and will result in more reliable and cost effective designs.

E. *Navigation Channels and Bank Protection*

In 70's it was decided to build a navigation channel (known as Schelde-Rhein connection) as a connection between Rotterdam and Antwerp. Such large project required proper information for design.

The water motion induced by ships can often lead to the erosion of the bank protection of ship canals. The problem becomes more serious nowadays because of recent developments in inland shipping involving the increasing engine power of the vessels and the introduction of push-tow units on smaller canals. A systematic fundamental research program on ship-induced water motion and the related bank protection design has resulted in design guide-lines based on small-scale hydraulic models. Small-scale tests however, only yield limited information.

Figure 13. Prototype tests Hartel Canal

Therefore, it was decided to carry out two series of full-scale measurements in Hartel canal to verify the results of the model tests (1981, 1983). In addition the influence of the subsoil on the stability of bank protection, which generally can not be studied in small-scale models, was studied at full scale [27,28].

F. Materials and Systems

The cost of production and transportation of materials required for hydraulic and coastal structures is an important consideration when selecting a particular design solution. Thus it is important to establish the availability and quality of materials for a particular site at an early stage when considering design options. Using the available tools and models, the structure can be designed to perform the functional requirements. An additional problem is that these functions will change with time in service because of material degradation processes. Therefore the designer's skill must also encompass consideration of durability and degradation processes. A degradation models for materials and structures should be developed so that the whole-life consequences may be considered at the design stage (NB. a provisional model for armour stone, which considered rock and environmental parameters, is presented in the CUR/CIRIA Manual [17].

Wastes and industrial by-products as alternative materials:

Domestic and industrial wastes and industrial by-products form a still growing problem especially in high-industrialized countries or highly populated regions. A careful policy on application of these materials in civil engineering may (partly) help to reduce this problem. Current European policies aim to increase the use of waste materials of all kinds and to find economic, satisfactory and safe means of their disposal. The use of waste materials in hydraulic and coastal structures is limited by their particle size distribution, mechanical and chemical stabilities and the need to avoid materials which present an actual or potential toxic hazard [17,18,29].

In the Netherlands, due to the lack of natural rock resources, the application of waste materials in civil engineering has already a long tradition. The large scale application of alternative materials started during execution of Delta works.

The extensive research on properties of waste materials allows making a proper selection depending on environmental requirements. Waste materials such as silex, quarry wastes, dredging sludge (depending on the source/location), and many minestone wastes have little or no hazardous contamination. These materials can be used as possible core, embankment fill or filter material. The engineering properties of many waste materials are often comparable or better than traditional materials. Slags have good friction properties due to their angularity and roughness and typically have high density. Mine wastes sometimes have poor weathering characteristics, but are usually inert and have satisfactory grading for deep fills. The fine materials such as fly ashes and ground slags are already in general use as cement replacement and fillers. Good quality control, not only for limiting the potential for toxic hazard, but also of the mechanical properties of waste materials can considerably increase the use of such low-cost materials in appropriately designed coastal and bank protection structures [29].

Geosynthetics and geosystems:

Geosynthetics are relatively a new type of construction material and gained a large popularity especially in geotechnical engineering and as component for filter structures. There is a large number of types and properties of geosynthetics, which can be tailored to the project requirements [26]. Geosynthetics have already transformed geotechnical engineering to the point that it is no longer possible to do geotechnical engineering without geosynthetics; they are used for drainage, reinforcement of embankments, reduction of settlement, temporary erosion control, and hazardous waste containment facilities. These latest are very often planned as land reclamation along the shores.

When geosynthetic materials or products are applied in civil engineering, they are intended to perform particular functions for a minimum expected time, called the design life. Therefore, the most common (and reasonable) question when applying geosynthetics is 'what is the expected/guaranteed lifespan of these materials and products'. There is no a straight answers to this question. Actually, it is still a matter of 'to believe or not to believe'. Both the experimental theory and practice cannot answer this question yet. However, the Dutch evaluation of the long-term performance of the older applications of geotextiles (back to 1968) has proved that the hydraulic functioning was still satisfactory. A similar conclusion has been drawn from the recent evaluation of the long-term performance of nonwoven geotextiles from five coastal and bank-protection projects in USA [30].

The technology of geosynthetics has improved considerably in the years. Therefore, one may expect that with all the modern additives and UV-stabilizers, the quality of geosynthetics is (or can be, on request) much higher than in the 60s. Therefore, for the 'unbelievers' among us, the answer about the guaranteed design life of geosynthetics can be at least 50 years. For 'believers', one may assume about 100 years or more for buried or underwater applications. These intriguing questions on the lifespan of geosynthetics are the subject of various studies and the development of various test methods over the world. Also, the international agencies related to normalization and standardization are very active in this field. The recent guide (European Standard) of the European Normalization Committee presents the actual 'normalized knowledge' on this subject (CEN/CR ISO, 1998). The object of this durability assessment is to provide the designing engineer with the necessary information (generally defined in terms of material reduction or partial safety factors) so that the expected design life can be achieved with confidence.

Geosystems: In recent years traditional forms of river and coastal works/structures have become very expensive to build and maintain. Various structures/systems can be of use in hydraulic and coastal engineering, from traditional rubble and/or concrete systems to more novel materials and systems such as geotextiles/geosynthetics, natural (geo)textiles, gabions, waste materials, etc. Moreover, there is a growing interest both in developed and developing countries in low-cost or novel engineering methods, particularly as the capital cost of defence works and their maintenance continue to rise. The shortage of natural rock in certain geographical regions can also be a reason for looking to other materials and systems. This all has prompted a demand for cheaper, less massive and more environmentally acceptable engineering. However, besides the standard application in filter constructions, the application of geosynthetics and geosystems in hydraulic and coastal engineering still has a very incidental character, and it is usually not treated as a serious alternative to the conventional solutions. That was for the author the main reason to write the state-of-the-art on application of geosynthetics and geosystems in hydraulic and coastal engineering [26].

These new (geo)systems (geomattresses, geobags, geotubes, seaweed, geocurtains and screens) were applied successfully in number of countries and they deserve to be applied on a larger scale. Recently, geocontainers filled with dredged material have been used in dikes and breakwaters in a number of projects around the world, and their use in this field is growing very fast. Also, a number of new applications for geosynthetic curtains and screens have been developed and tested in practice.

Because of the lower price and easier execution these systems can be a good alternative for protective structures in hydraulic and coastal engineering both in developed and developing countries. The main obstacle in their application, however, is the lack of proper design criteria (in comparison with rock, concrete units, etc.). In the past, the design of these systems was mostly based on rather vague experience than on the general valid calculation methods. More research, especially concerning the large-scale tests and the evaluation of the performance of projects already realised, is still needed. In Reference [26] an overview is given of the existing geotextile systems, their design methods (if available), and their applications. Where possible, some comparison with traditional materials and/or systems is presented. The recent research on some of these systems has provided better insight into the design and applications.

G. Technology Transfer

Know-how/technology transfer is an important expedient in the sustainable development of nations. Technology transfer is sustainable when it is able to deliver an appropriate level of benefits for an extended period of time, after major financial, managerial and technical assistance from an external partner is terminated. Apart from clearly identified objectives for Technology Transfer projects, proper project design and well-managed project execution, essential factors conditioning the survival of projects include: policy environment in recipient institution/country, appropriateness of technology and management organisational capacity.

Technology Transfer means the transfer of knowledge and skills, possibly in combination with available tools, to institutions and individuals, with the ultimate aim to contribute to the sustainable development of the receiving institution (national) or country (international). Professional educational institutes are likely to restrict their work to specialised education and training of individuals. Institutions, however, generally aim at a broader approach. The objective of Technology Transfer should be to reinforce the capability of institutions and individuals to solve their problems independently. The required support can be indicated in a diagram differentiating between the analysis and solution phases of engineering problems. Obviously, knowledge and experience is required on three levels to obtain optimum results:

- knowledge of the processes
- knowledge and experience in the use of process simulation techniques
- experience in practical applications

Knowledge and experience can best be transferred in phases during a project that runs over several years. In many cases, however, budget restrictions call for another approach. The advisor may be called upon as a consultant, and the project includes only some of the phases mentioned above.

Not only knowledge and skills must be transferred to the client’s staff, but it must also be integrated in the client’s organization for future, independent use. In order to guarantee an efficient interaction between the transfer and the integration activities, distinction in phases is required. One may distinguish the following realization phases:

1. professional education (general scientific/technological level)
2. professional training (new/specific technology/skills/tools)
3. development phase (physical/logistic adjustments at recipient party)
4. institute support (advisory services/exchange visits during project)

The necessary number of phases may vary depending of situation (country, type of project).

Hydraulic and Coastal engineering is a complex art. At this moment a limited number of phenomena can be understood with the help of the laws of physics and fluid mechanics. For the remainder, formulas have been developed with a limited accuracy. In addition, input data are limited availability and form another source of uncertainty. Consequently, a sound engineering approach is required, based on practical experience and supported by physical and numerical models, to increase the understanding of many phenomena and to come up with sustainable solutions. Especially, standard’ solutions do not exist in coastal engineering; solutions very much depend on the local circumstances as well as the social and political approach towards coastal engineering. Consequently, the transfer of coastal engineering knowledge is a complex art as well.

Capacity building is important pre-condition for the realization of future challenges and transfer of know-how, especially for developing countries. Sustainable transfer of hydraulic and coastal engineering technology at post-graduate level should therefore aim at increasing the capacities and skills of the engineers such that they are able to analyse a problem correctly and identify possible directions of solutions.

In solving these problems, one should always analyse the cause of the problem. Sometimes it is easier to change something in the estuary or river, than to combat the erosion. When it is not possible to take away the cause of the problem, then a number of technical tools are available as discussed in this book. In the design of these methods in most cases a low-investment approach can be followed. Low investment solutions generally require more maintenance than capital-intensive solutions. Therefore the construction has to be designed in such a way that maintenance can be performed easily, with local means, thus, with local materials, local people and with local equipment. These requirements are not very special and one can meet these requirements easily. The main problem is that one has to realise these points during the design phase.

We may conclude from the above that modern engineer must be educated in various technical and non-technical fields. The task of engineers involved in problems of developing countries should be the adaptation (translation) the actual knowledge to appropriate technologies suitable for their problems and their possibilities.

V. Conclusions

Large projects (likes Delta Works) need usually some specific solutions. It stimulates new research and innovation, which contribute strongly to new developments in hydraulic and coastal engineering.

Problem identification and understanding is very important for a proper choice of solution to water (flood) management and coastal problems. Generally, it may be concluded that there are both physical as well as social aspects to every problem. As a consequence mere technical solutions often turn out to be mistake. Proper quality of environmental boundary conditions defines the quality of design.

There are a large number of hydraulic and coastal structures. For some of them, workable design criteria have been developed in recent years (rubble-mound breakwaters, riprap, block revetments, filter structures, etc.). However, many of these criteria/formulae are still not quite satisfactory, mainly because they are lacking physical background, what makes extrapolation beyond the present range of experience rather risky. To solve this problem, it will be necessary to continue physical model experiments (on scale and in prototype) to develop, validate and calibrate new theories. Moreover, there are still a large number of systems with not adequate design techniques, for example, groins, submerged/reef breakwaters, a number of revetment types (gabions, geomattresses), geosystems, open filter, prediction and measures against scouring, etc. However, opposite to the functional design, the structural design can always be solved by the existing means (design criteria if available or model investigation), assuming availability of funds. Functional design (especially for coastal problems) is one of the most important and most difficult stages in the design process. It defines the effectiveness of the measure (project) in solving specific problem. Unfortunately, there are still many coastal problems where the present functional design methods are rather doubtful, especially concerning shoreline erosion control measures (i.e. groins, sea walls). Also, adequate measures against lee-side erosion (flanking) deserve more attention.

Alternative (waste) materials and geosynthetics and geosystems constitute potential alternatives for more conventional materials and systems. They deserve to be applied on a larger scale. The geosynthetic durability and the long-term behaviour of geosystems belong to the category of overall uncertainties and create a serious obstacle in the wider application of geosynthetics and geosystems and, therefore, are still matters of concern.

The understanding of the coastal responses in respect to the sedimentary coast and its behaviour is at least qualitatively available. However, reliable quantification is still lacking which make functional design of shore erosion control structures very risky. Much mathematical and experimental work is still to be done. Because of scale effects, the experiments will have to be carried out in large facilities or may be verification is even only possible on the basis of prototype observations over a long period. This work is so complicated that international co-operation is almost a prerequisite to achieve success within a reasonable time and cost frame.

Research on hydraulic and coastal structures should benefit from more co-operations among researchers and the associated institutions. Publishing basic information and standardised data would be very useful and helpful in establishing a more general worldwide data bank available, for example, on a website. Systematic (international) monitoring of realised projects (including failure cases) and evaluation of the prototype and laboratory data may provide useful information for verification purposes and further improvement of design methods. It is also the role of the national and international organisations to identify this lack of information and to launch a multiclient studies for extended monitoring and testing programmes, to provide users with an independent assessment of the long-term performance of hydraulic and coastal structures, including alternative materials and systems (geosynthetics, geosystems, alternative materials, etc.).

Inventory, evaluation and dissemination of existing knowledge and future needs, and creating a worldwide accessible data bank are urgent future needs and some actions in this direction should be undertaken by international organisations involved. It should be recommended to organise periodically (within time span of 5 to 10 years) state-of-the-art reports on various sub-items of hydraulic and coastal engineering, which should be prepared by international experts in a certain field. It can be organised by creating semi-permanent working groups on specific subject and their activities should be paid from a common international fund, which should be established by one of the international organization.

Adjustment of the present education system as a part of capacity building for solving future problems should be recognised as one of the new challenges in hydraulic and coastal engineering. It is also important for the proper technology transfer to developing countries and development and maintaining of appropriate technologies for local use. Moreover, solidarity must be found in sharing knowledge, costs and benefits with less developed countries which are not able to facilitate the future requirements of integrated coastal management by themselves (on its own) including possible effects of climate change. More attention should be paid to integration of technological innovation with institutional reforms, to rise of awareness to change human behaviour, to developing of appropriate technologies that are affordable by poorer countries, to promoting technologies that would fit into small land holdings (local communities), and to capacity building (education), which is needed to continue this process in sustainable way.

Finally, there is a continuous development in the field of hydraulic and coastal engineering, and there is always a certain time gap between new developments (products and design criteria) and publishing them in manuals or professional books. Therefore, it is recommended to follow the professional literature on this subject for updating the present knowledge and/or exchanging new ideas.

REFERENCES

[1] Pilarczyk, K.W., ed., 1998, Dikes and Revetments, *A.A. Balkema Publisher*, NL

[2] RWS, 1994. DESIGN PLAN OOSTERSCHELDE STORM-SURGE BARRIER: Overall Design and Design Philosophy, *Ministry of Transport, Public Works and Water Management, Rijkswaterstaat (RWS)*, (A.A. Balkema, Publ.), Netherlands.

[3] RWS (Rijkswaterstaat), 1984, 1987. The Closure of Tidal Basins; closing of estuaries, tidal inlets and dike breaches, published by the *Delft University Press*, Delft, NL.

[4] RWS, 1980, HYDRAULIC ASPECTS OF COASTAL STRUCTURES: Developments in Hydraulic Engineering related to the design of the Oosterschelde Storm Surge Barrier in The Netherlands, *Delft University Press*, Delft.

[5] D'Angremond, K. and Van Roode, F.C., 2001, Breakwaters and closure dams, *Delft University Press*, Delft, the Netherlands.

[6] CUR, 1992, Sand Closures, *Centre for Civil Engineering Research and Codes, Report 157*, P.O. Box 420, Gouda, NL.

[7] Breusers, H.N.C., 1966, Conformity and time scale in two-dimensional local scour, Proceedings Symposium on model and prototype conformity, *Hydraulic Research Laboratory, Poona*.

[8] Breusers, H.N.C. and A.J. Raudkivi, 1991, Scouring, Hydraulic structures design manual, *IAHR*, A.A.Balkema, Rotterdam.

[9] De Graauw, A.F.F., and K.W. Pilarczyk, 1981, Model-prototype conformity of local scour in non-cohesive sediments beneath overflow-dam, *19th IAHR-congress*, New Delhi (**also** Delft Hydraulics, Publication No.242)

[10] Hoffmans, G.J.C.M., 1992, Two-dimensional mathematical modelling of local-scour holes, Doctoral thesis, *Faculty of Civil Engineering, Hydraulic and Geotechnical Engineering Division, Delft University of Technology*, Delft.

[11] Hoffmans, G.J.C.M. and K.W. Pilarczyk , 1995, Local scour downstream of hydraulic structures, *Journal of Hydraulic Engineering, ASCE*, Vol. 121, No.4, pp. 326-340.

[12] Hoffmans, G.J.C.M. and H.J. Verheij, 1997, Scour Manual, *A.A. Balkema, Rotterdam, The Netherlands*

[13] Van der Meer, J.P., 1988, Rock slopes and gravel beaches under wave attack, PhD-thesis *Delft University of Technology*, April 1988; (also available as *Delft Hydraulics Communication* 396).

[14] Ahrens, J.P., 1975, Large Wave Tank Tests of Riprap Stability, *C.E.R.C. Technical Memorandum no. 51*, May.

[15] Pilarczyk, K.W., 1984, Interaction water motion and closing elements, The closure of tidal basins, p.387-405, *Delft University Press*, Delft.

[16] Van Hijum, E. and Pilarczyk, K.W., 1982, Gravel beaches; equilibrium profile and longshore transport of coarse material under regular and irregular wave attack, *Delft Hydraulics Laboratory, Publication no. 274, July*, Delft, the Netherlands.

[17] CUR/CIRIA, 1991, Manual on use of rock in coastal engineering, CUR/CIRIA report 154, *Centre for Civil Engineering Research and Codes (CUR),* Gouda, the Netherlands.

[18] CUR/RWS, 1995, Manual on the use of rock in hydraulic engineering, CUR report 169, *Centre for Civil Engineering Research and Codes (CUR)*, P.O. Box 420, 2800 AK Gouda, A.A. Balkema (Publisher), NL.

[19] Van Gent, M.R.A., 1995, Wave interaction with permeable coastal structures, Ph.D.-thesis, *Delft University of Technology, ISBN 90-407-1182-8, Delft University Press.*

[20] De Groot, M.B., Bezuijen, A., Burger, A.M. and Konter, J.L.M., 1988, The interaction between soil, water and bed or slope protection. *Int. Symp. on Modelling Soil-Water-Structure Interactions*, Delft, A. Balkema.

[21] Burger, A.M., M. Klein Breteler, L. Banach, A. Bezuijen and K.W. Pilarczyk, 1990; Analytical design formulas for relatively closed block revetments; *Journal of Waterway, Port, Coastal and Ocean Eng., ASCE*, vol. 116 no. 5, Sept/Oct.

[22] PIANC, 1992, Analysis of rubble mound breakwaters, Report of Working Group 12, *PIANC, Supplement to Bulletin No. 78/79,* Brussels, Belgium.

[23] ASCE, 1995, Wave Forces on Inclined and Vertical Wall Structures, *Task Committee on Forces on Inclined and Vertical Wall Structures, ASCE*, New York.

[24] *CUR* 1995, Design manual for pitched slope protection, *CUR report 155, ISBN 90 5410 606 9,* Gouda, the Netherlands.

[25] Schiereck, G.J., 2001, Introduction to bed, bank and shore protection, *Delft University Press,* Delft.

[26] Pilarczyk, K.W., 2000, Geosynthetics and Geosystems in Hydraulic and Coastal Engineering, *A.A. Balkema*, Rotterdam.

[27] Blaauw, H.G., M.T. de Groot, F.C.M. van der Knaap and K.W. Pilarczyk, 1984, Design of bank protection of inland navigation fairways, *in Flexible Armoured Revetments incorporating geotextiles,*Thomas Telford Ltd, London.

[28] PIANC, 1987, Guidelines for the design and construction of flexible revetments incorporating geotextiles for inland waterways, *PIANC, Supplement to Bulletin No. 57,* Brussels, Belgium.

[29] Pilarczyk, K.W., Laan, G.J. and Den Adel, H., 1987, Application of some waste materials in hydraulic engineering, *2nd European Conference on Environmental Technology*, Amsterdam.

[30] Mannsbart, G. and B.R. Christopher, 1997, Long-term performance of nonwoven geotextile filters in five coastal and bank protection projects, *Geotextiles and Geomembranes,* Vol. 15, no. 4-6.

WEBSITES: Publications online:

www.tawinfo.nl
(select English, downloads)
http://www.wldelft.nl/rnd/publ/search.html
http://www.waterbouw.tudelft.nl/
http://www.delftcluster.nl/
http://www.unesco-ihe.org/we/dicea/int03/default.htm?http://www.unesco-ihe.org/we/dicea/int03/int0301.htm
http://mail.vssd.nl/hlf/f011contents.pdf
http://www.waterbouw.tudelft.nl/public/verhagen/

Storm-surge barries Eastern Scheldt:
http://www.deltawerken.com/English/10.html?setlanguage=en
http://www.deltawerken.com/Deltaworks/23.html

Storm-surge barrire Rotterdam:
http://www.keringhuis.nl/engels/home_flash.html

http://www.antiqbook.nl/boox/nij/14953.shtml:
HUIS IN 'T VELD, J.C. [ET AL.]. **The Closure of Tidal Basins. Closing of Estuaries, Tidal Inlets and Dike Breaches;**
Delft University Press. 1984, 1987 (sec.ed); ISBN: 90-6275-287-x.
Delft University Press: info@library.tudelft.nl;
http://www.library.tudelft.nl/ws/index.htm

Key Words: Delta Works, Floods, Closures, Scour, Revetments, Hydraulic Design.