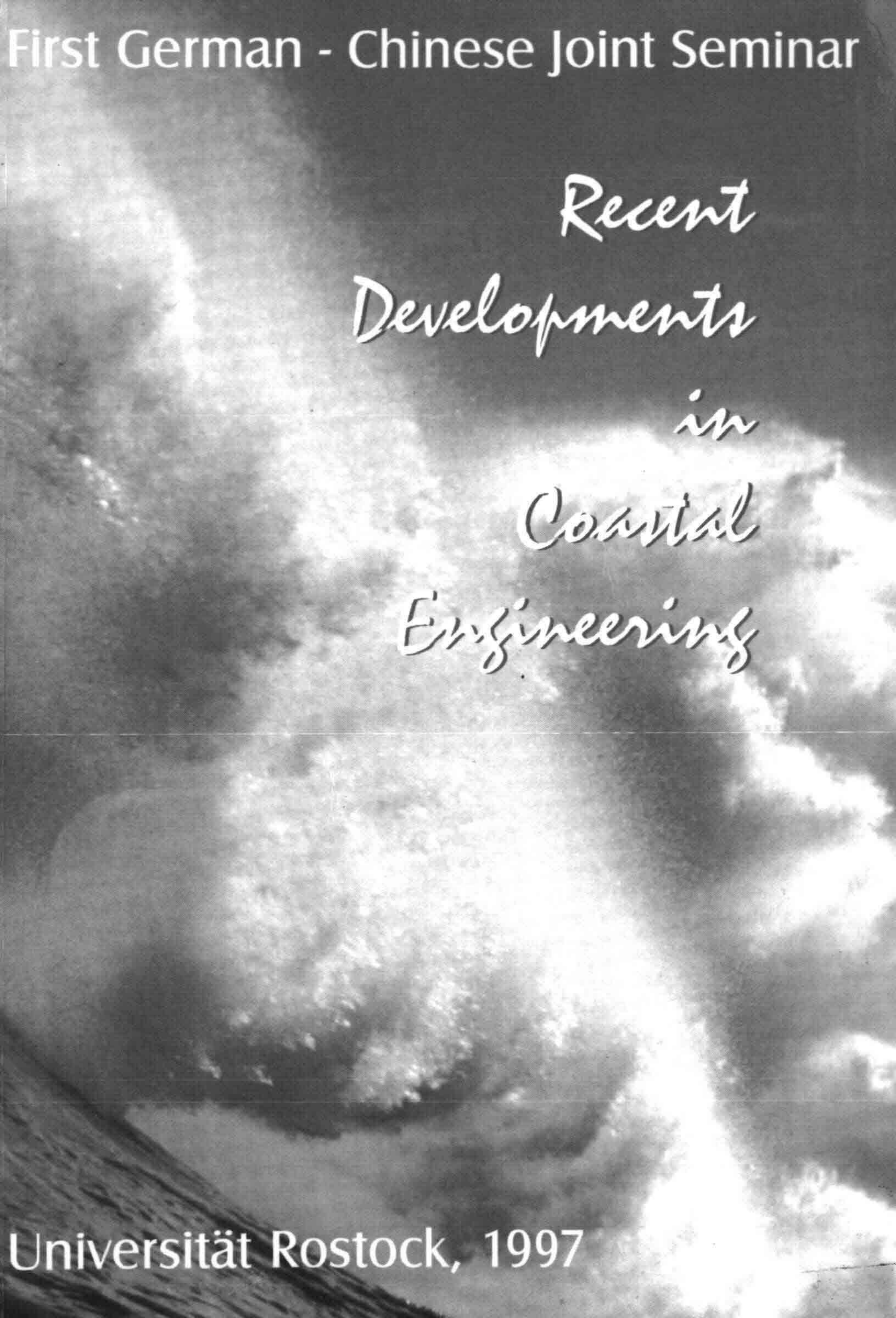
First German - Chinese Joint Seminar



Recent Developments in Coastal Engineering

Universität Rostock, 1997

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Institut für Wasserbau

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College of Engineering

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Hasenwinkel, September 8th to 10th 1997

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Preface

The First German-Chinese Joint Seminar on Recent Developments in Coastal Engineering was planned on a communal basis between the University of Rostock and the National Cheng Kung University of Tainan. It was held from 08.09 - 12.09.1997 with the aim of increasing collaboration between university institutes and specialist authorities in Germany and Republic of China in the field of coastal engineering, as well as strengthening the basis for the economic and technical cooperation of both countries in the future.

Although there have been close links between universities in Germany and Republic of China for over 20 years in the field of coastal engineering, they have more or less been limited to personal contact between individual scientists.

Due to the bilateral agreements on scientific and technical cooperation which have existed for some time between the German Research Community (DFG) and the National Science Council (NSC) of the Republic of China, it was possible to bring together scientists and experts from practical and administrative fields in the picturesque environment of one of the beautiful castles in Mecklenburg-Vorpommern, where they were able to have intensive discussions about present matters of interest in coastal hydraulic engineering.

The report presented here about the seminar on recent developments in coastal engineering, which was held within the framework of the symposium, reproduces the lectures which were given by the German and Chinese delegates in the course of the seminar.

The contributions, which are partly presented in an extended form, give an impression of the main points of content of the seminar, which was attended and debated in a lively fashion by the 50 or so experts invited. An excursion was organized by the University of Rostock for the delegates from Republic of China after the seminar, so as to give the Taiwanese guests an insight into matters of topical importance in Germany.

Important outcomes of the seminar are not least the agreements on a deepening of the cooperation begun with the symposium, whereby the main points of content were based on seminars and discussions which could not be repeated here.

There were intensive discussions about the possibilities of restructuring the framework of existing agreements on cooperation by individual participants. These have already led to concrete proposals just months after the seminar ended.

The organizers would like to thank all patrons and sponsors of the symposium for their support, without which the event would not have been either financially or organizationally possible. Our thanks goes to the University of Rostock and especially the DFG and the NSC, who paid for the largest part of the not inconsiderable costs incurred by travel, accommodation and catering expenses.

We would like to thank all those who contributed to the event's success: the honourable guests for their words of greeting during the opening speeches, the University of Rostock for its considerable interest in the event itself and its financial help in the printing of this report. We would also like to thank the participants in the seminar for their written and oral contributions, and the regional and Federal institutions involved in the excursion for their organizational assistance. Finally our thanks go to those engineering bureaus and building firms which showed an interest in the event and contributed to it financially.

The report should document the successful event, while at the same time giving an insight into the form of cooperation to those interested colleagues who were unable to take part in the event, due to the limited number of participants.

Chia Chuen Kao

Sören Kohlhase

Organisation

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  Sören Kohlhase (Rostock), Peter Petersen (Kiel)

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Opening Session

- Welcome Addresses

Prof. Dr. sc. nat. Gerhardt Maeß, Rektor der Universität Rostock

Günther Leymann, Ministerialdirigent, Ministerium für Bau, Landesentwicklung und Umwelt, Mecklenburg-Vorpommern

Hermann Fischer, Ministerialdirigent, Kultusministerium des Landes Mecklenburg-Vorpommern

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Prof. Dr.-Ing. Otto Fiedler, Fakultät für Ingenieurwissenschaften, Universität Rostock

Prof. Dr. Robert R. Hwang, Academia Sinica, Leader of the Chinese Delegation

- Invited Speaker

Dr. rer. nat. Rolf-Peter Randl, Ministerialrat, Bundesministerium für Bildung, Wissenschaft, Forschung und Technologie

- Introduction to the Seminar

Prof. Dr.-Ing. Chia-Chuen Kao, National Cheng-Kung University, Tainan

Prof. Dr.-Ing. Sören Kohlhase, Universität Rostock

The Purpose of Coastal and Marine Research in Germany

Rolf Peter Randl

Federal Ministry for Education, Sciene, Research and Technology

Isn't it a strange title? Everybody knows the reasons for coastal and marine research. Naturally, we intend to care for the preservation of the coast line, furthermore we need to save mankind from the detrimental impacts from storms, tidal influence and the rising sealevel.

That sounds as if we fight fearlessly in front place with a touch of unselfishness and heroism. Let me be a little bit cynical: this sounds great but is nevertheless totally untrue.

To follow the reality: Germany has a long standing record of successful dike wardens, is well reknown for a proper handling of coastal management, for the ability and thoroughness of the respective administations. That's the reason why -when coming to the retirement- the typical german coastal guard starts a new career: too busy to be able to compile his numerous experiences gained in his professional life, he now has to care that these experiences are compiled and collected for the sake of his successors: it is absolutely clear that the NIH-principle does not play a role, not everybody must duplicate the mistakes already made but should share in the propects of a lifetime's experience. For the sake of clarity: NIH reads "Not invented here". And the successor in the job gaines -after a while of several years and out of the shadows of a personal memory- all knowledge gathered by his ancestor -over generations and generations- with the bright know-how over about l00m of shorelength.

Not being bred as coasterner, sentenced by heritage to be a coastal protector, I observe some strange habits in the coastal-management-community. I feel it interesting to know all details of a specific bight, but that must not be explored in a broad research and development program if there is no chance to gain from this a better understanding of a longer coastline as such. From my education, it is necessary to understand behaviour as a systems approach in order to facilitate proper action. By the way: before I became civil servant in research management, I have been a nuclear chemist.

So, when I took over the responsibility in this area, I -and naturally with the help of two collegues in the field- I turned from the sterile research of minute problems, e.g. to get acquainted with the christian names of each sand pebble of 10cm of shore length, to a more systematic approach for the problems which exist.

On the german coast areas of the Northern Sea and the Baltic, the responsibility of guarding and keeping the coastal zones and the adjacent oceans lies with the respective special administrative bodies of the federal and state governments. They have to fulfill their duties according to the law. Their duties vary considerably as e.g. navigation, administration of ports, coastal management etc. and their work loads are subdivided regionally. Very often, the management tools are the same, based on comparable technical and scientific terms. In order to optimize the advantages and to minimize costs, the administration cooperate together and support each other.

A legal framework for this cooparation is Paragraph 91 b of the constituency, defining a Federal-States relationship as regards the coastal protection. In terms of research and development, this calls for research results close to practical needs, the stimulation of harmonized work with university basic ideas and the cooperation and information exchange internationally, especially within Europe. For this purpose, an administrative body was founded, the "Trust for research in coastal management". Members are the german coastal states and the federal ministries of agriculture, transportation and technology in their respective duties. And to add from before, the people who helped me a lot in approaching the ongoing research more from a systems approach point of view were the operating director of the Trust and its research director.

The german coast line is usually composed of sandy beaches. Not taking into account in the first instance influences of wind and waves, the main difference lies in the occurance of tides: the Northern Sea shores are influenced by tides, the Baltic ones aren't. Furthermore, a distinction in the Baltic has to be made regarding the area: in Schleswig-Holstein, we rely mainly on dikes whereas in Mecklenburg-Vorpommern the choice was made to have beach forests to beware from floods. Another very typical distinction is the shallow water area directly in front of the shores. Here, the Northern Sea presents the wadden see contours, the Baltic doesn't. To describe the research in a morphological sense, this overall presentations show the basic roads.

Let's come to the main features for additional influences: the behaviour of the tidal attack on the shores, the winds as regards not only the effects of storm events on the shores but also the frequency of events, wind direction, not to forget aeolic sand transport. Additionally we try to understand the situations in estuarine areas since in the mouth of our big rivers we usually have heavy commercial marine transportation.

One task is somewhat extraordinary in nature: coming from a virtual university cooperation concept, it is anticipated to combine this with the administration's actual needs for a specific question, so that researchers at university are directly linked to administrations data and vice versa. Taking into acount the most unbelievable differences in thinking structures, a positive result here would be a break-through.

The foregoing describe the present basic efforts in a systematic approach to coastal management. We are running these days about two dozen projects and hope we will gain results applicable and helpful for the practical engineer in duty. Upon request I can describe to you single R&D programs individually outside this presentation.

I furthermore restricted myself in describing only coastal management R&D. I purposly omitted the broad area of marine research from a pure scientific point of view as there are polar research, deep sea, international tasks as "GOOS" etc. which naturally has also its place in the research program. If you wished to obtain more information about these topics, please let me know, I will care for it.

Thank you for your attention.

Topic I

Coastal Protection

Marine Environment and Global Changes

Chairman: Hocine Oumeraci

COASTAL PROTECTION AND ITS MANAGEMENT IN GERMANY

Heie F. Erchinger
state coast protection agency, Norden

Abstract

The north coast of the Federal Republic of Germany is bounded by two completely different types of sea, the North Sea adjoining the Atlantic Ocean with pronounced tidal action and frequent storm tides, and the almost completely enclosed Baltic Sea with almost negligible tidal action. Accordingly, the character and protection of these two stretches of coastline are vastly different.

Coastal Protection is concerned with the following tasks

- to protect people, their living spaces, their bases of existence and the associated infrastructure from the destructive effects of coastal erosion and flooding due to storm tides;
- to maintain and in some cases to manage protective structures and facilities;
- to organise storm tide warning services and disaster control services.

Coastal protection is the reponsibility of states bordering the coastline. 70 % of the costs for providing sufficient coastal protection structures is derived from the federal government's community task funds. A prerequisite for the design and constructions of coastal protection structures is a study of morphological and hydrological conditions as well as dynamic loading, especially due to wave action. Basic design planning and legal regulations are the responsibility of the water recources or hydraulic engineering administrations of the respective coastal states. The public authorities of these states are also generally reponsible for carrying out construction work. The federal states are also responsible for the maintenance and operation of coastal protection structures and facilities. The maintenance of the main dykes on the mainland coast of two federal states is carried out almost exclusively by the Dyke Associations, comprised of owners of the regions endangered by storm tides. The storm tide warning service is again the responsibility of the respective states whereas the disaster control service is under the jurisdiction of rural districts.

Extensive stretches of the coastline lie within local national parks. The undisturbed and completely natural development strived for in the national parks is not always in keeping with coastal protection interests. Compromise

solutions must therefore be agreed upon between these two spheres of responsibility. More recently, these are drawn up in management plans for the stretches of coastline concerned.

1 The coasts of Germany, the geographic and hydrologic situation

The north of the Federal Republic of Germany is locally bounded by two very different seas: the North Sea as a marginal sea of the Atlantic Ocean with distinct tidal action and frequent high storm surges and the almost secluded Baltic Sea which only has a neglectible low tidal activity. In correspondence, there are considerable differences between the character and the protection of these two coasts (fig. 1).

fig.1: Map of the German coasts at the North Sea and Baltic Sea with the adjacent federal states.

The tidal flat is located in front of the German North Sea coast. This 5 to 20 km wide amphibious landscape gets dry to 70% every day and has an especially high ecological value. Its outer fringe is bordered by a chain of sandy islands in the western and northern section. The coast land which is next to the tidal flats consists of low and absolute even marshland of maritime deposits of the Holocene. The average tidal range in the run of the German North Sea coast is 2,0 to 3,6 m.

The micro tidal coast of the Baltic Sea consists of flat and steeply coasts with numerous firths and bays and several rivers which flow into these. The adjacent land consists of deposits from the Ice Age with a hilly surface.

The German coasts are located in the zone of the predominant west winds due to their geographic position on approximately 55° northern latitude. Storms which come from a southwest to northwest direction sometimes become violent at the North Sea coast and often cause very heavy tidal storm surges that can lead to highest high water levels of 3,5 to 4,5 meters over MHWL and heavy sea state due to the bay effect. On February 16./17.1962 more than 300 people died in the hurricane flood (fig. 2). On the gauge record it can be seen, that at the port of Wilhelmshaven the storm surge water level was 3,7 m higher than the astronomical tide. The frequency of storm surges with a congestion of 1,5 m and more increased in the last 40 years, for example on the island Norderney from 1,6 to 2,5 yearly about 56% (fig. 3).

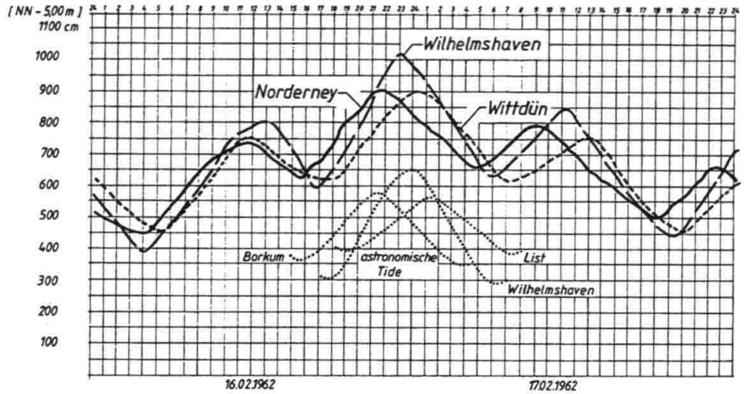


fig.2: Gauge record after the hurricane tide on the 16th/17th of February 1962 for Norderney, Wilhelmshaven and Wittdün [5].

It is far more rare that the German Baltic Sea is endangered by floodwater. This only happens when storms appear from northern to eastern directions and it possibly can be on a high level for a couple of days. The heaviest storm surge in the Baltic Sea was recorded in 1872 with peaks which increased from 2,5 to 3,5 m over the MW-level.

2 Coastal protection - protection of storm surges and coastal recession

The protection of coasts is a public task and contains

- the protection of the people, their living space, their basis of existence and the appertaining infrastructure against destruction by tidal refraction and the protection from overflowing storm surges.
- the planning, creating, maintaining and managing of protection constructions and facilities.
- the organization of a storm search warning service and high storm water disaster defense

The different facts and pretensions require the following protection constructions and provisions: The sandy islands on the outer tidal flat base are protected from overflowing floods by shores and chains of dunes at the seaside At the southern North Sea coast the longshore transport dominates.. The straits between the single barrier islands are overcome by bars which are formed to a northward oscillated arch by the strong ebb current. The accretion point of this bar arch on the next island is very important for its sand supply (fig. 4). To the east you have satisfactory sand supply and wide beaches and natural dunes. Sand gathering by brushwood fences and dune stabilizing and strengthening by planting beach grasses are possible. On the other hand you have beaches with constant erosion westward of this point of approach.

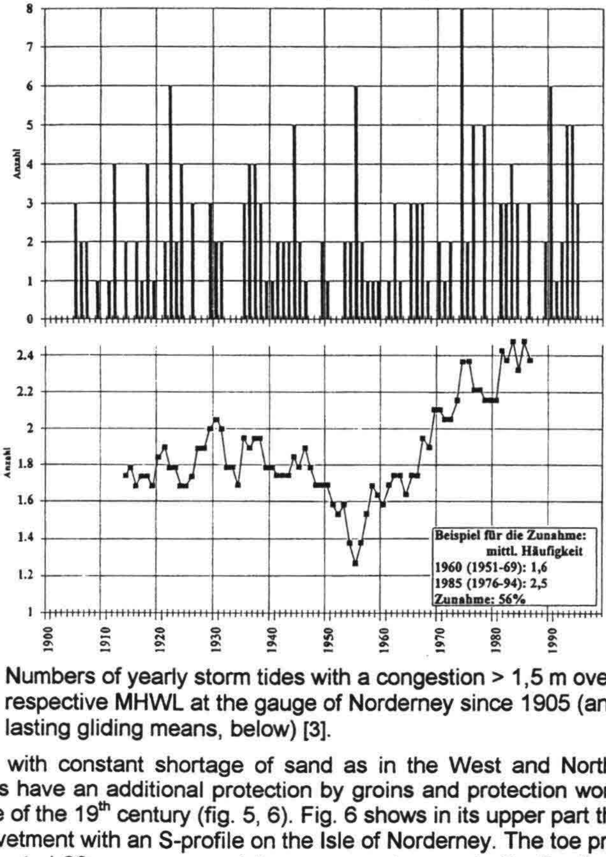


fig. 3: Numbers of yearly storm tides with a congestion > 1,5 m over the respective MHWL at the gauge of Norderney since 1905 (and its 19 year lasting gliding means, below) [3].

Areas with constant shortage of sand as in the West and Northwest of the islands have an additional protection by groins and protection works since the middle of the 19th century (fig. 5, 6). Fig. 6 shows in its upper part the 130 years old revetment with an S-profile on the Isle of Norderney. The toe protection was constructed 60 years ago and the upper part was rebuilt after the 1962 storm surge. Since 1951, beach nourishments were done for the protection of eroding beaches and dunes. It is necessary to do this on the islands Norderney and Langeoog at the East Frisian and Sylt at the North Frisian coast. The 40 km long west coast of Sylt requires the highest expenditure for its safety with about 2 Million m^3 sand volume yearly.

Another solution we have fixed upon on the Isle of Langeoog: After a long time of erosion of the barrier dune a backward dune strengthening will be carried out. It is a sort of retreat. It should be combined with a beach nourishment as the fresh water supply is hardly endagered (fig. 7).

The flat and low lying land along the mainland coast of the North Sea and the discharging tidal rivers are protected from storm surges by a continuous barrage: With an experience of almost 1000 years, the residents at the coast have developed the present flatly inclined dike sections which are up to 9 m high and have a basic breadth of 100 m. They are formed for the defense of the high storm water levels and substantial wave run ups normally as grassed dikes from a sand fill with a 1 to 2 m thick clay surface and they have to withstand the heavy swell load (fig 8).

Main dikes are also built at the backsides of the islands many times for the protection of low located parts of the islands with settlements and infrastructure constructions. The dikes and dunes for protection (dunes that have the function of a dike) form the surrounding ring of protection of an island.

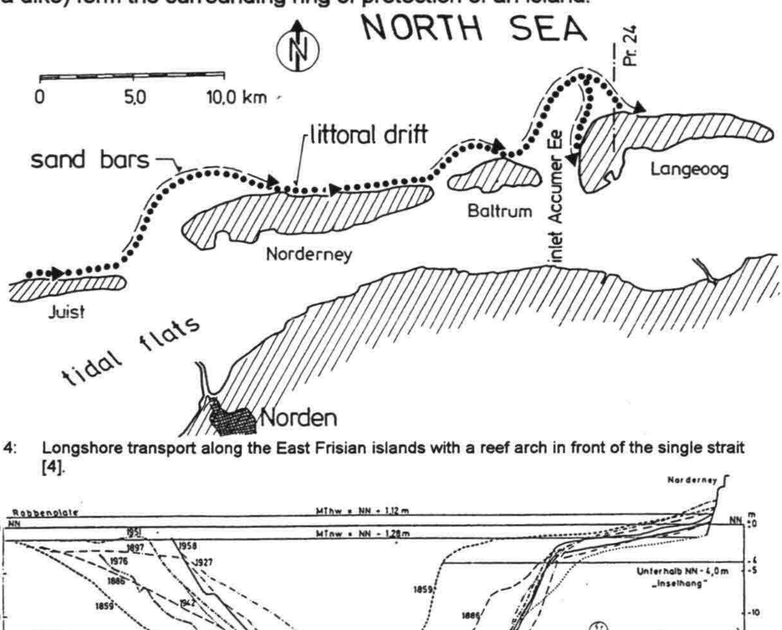


fig. 4: Longshore transport along the East Frisian islands with a reef arch in front of the single strait [4].

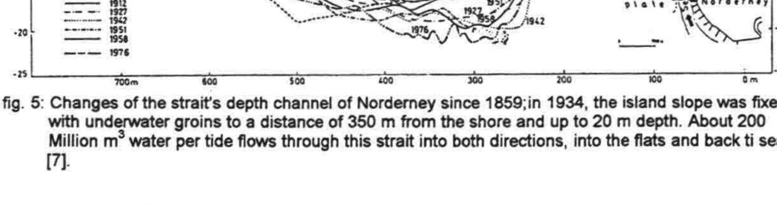


fig. 5: Changes of the strait's depth channel of Norderney since 1859;in 1934, the island slope was fixed with underwater groins to a distance of 350 m from the shore and up to 20 m depth. About 200 Million m^3 water per tide flows through this strait into both directions, into the flats and back ti sea [7].

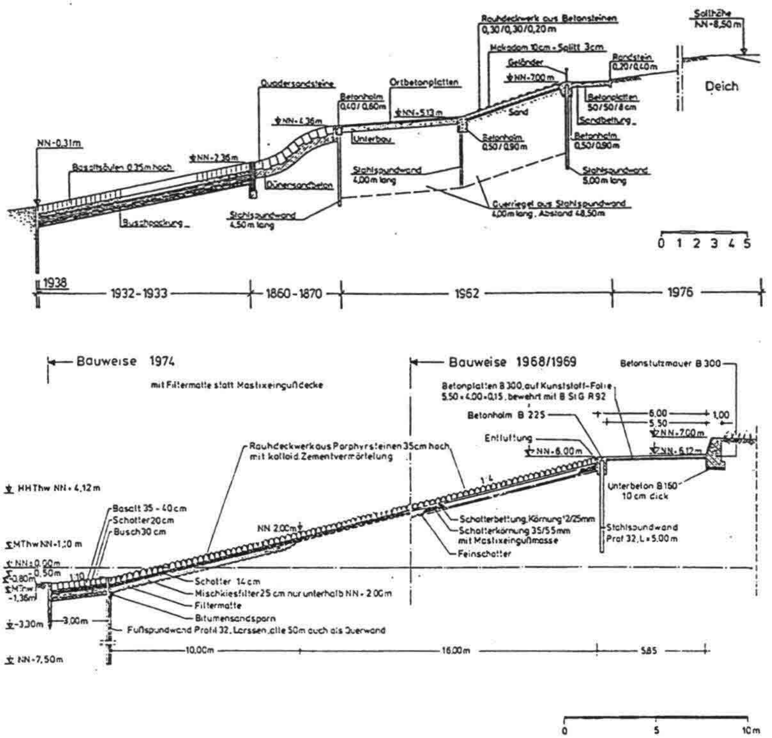


fig. 6: Cross sections of dune protection works on the East frisian islands Norderney and Baltrum (below) [1].

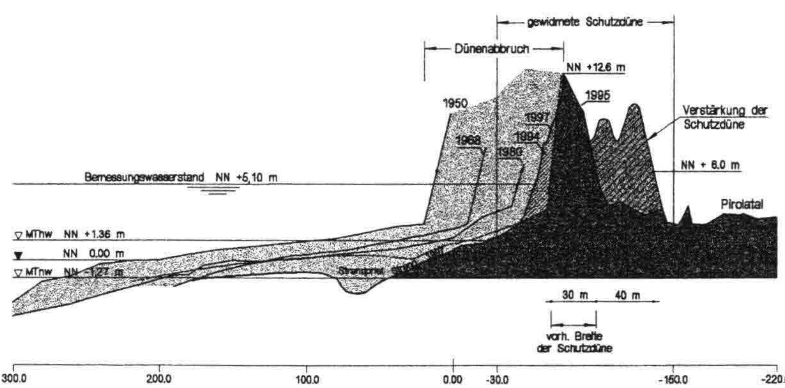


fig. 7:Cross section of the barrier dune on the Isle of Langeoog with severe dune and beach erosion since 1950 and a backward dune strengthening

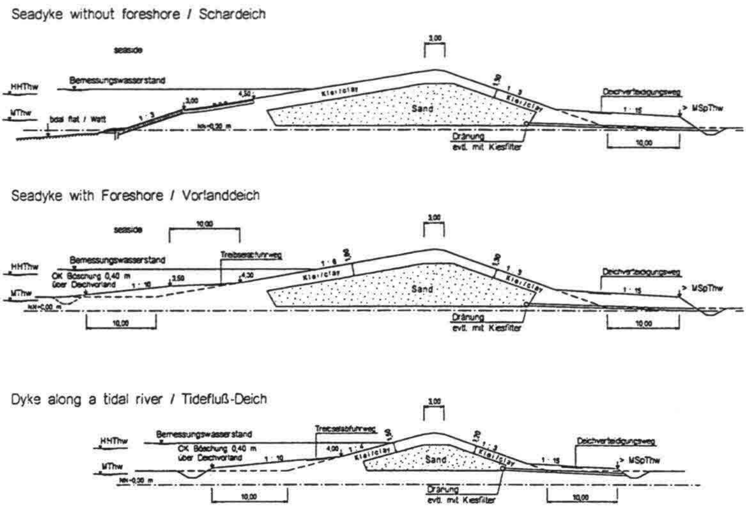


fig. 8: Cross- sections of sea dikes and river dikes [2]..

Dikes with foreland don't need a special toe construction. A 200 to 600 m wide dike foreland in front of the dike will be preserved in order to absorb the sea state, to cause a lower wave run up and to improve the dike protection. (fig. 9). Several flat inclined low „summerdikes" have been built on broad foreland for serving the further absorption of the waves and for additionally protecting the dikes.Therefore the following protection barriers also belong to the coastal protection system:

- the dike foreland including possible summerdikes
- the high located tidal flat areas
- the bars and shores as high and wide as possible

The lengths of the main dikes at the North Sea coast in the four Federal states Niedersachsen, Schleswig-Holstein, Hamburg and Bremen are in total about 1300 km (table 1).

Barrages in the course of the main dike line block numerous tidal rivers respectively their tributary rivers against storm surge water levels (fig.12).

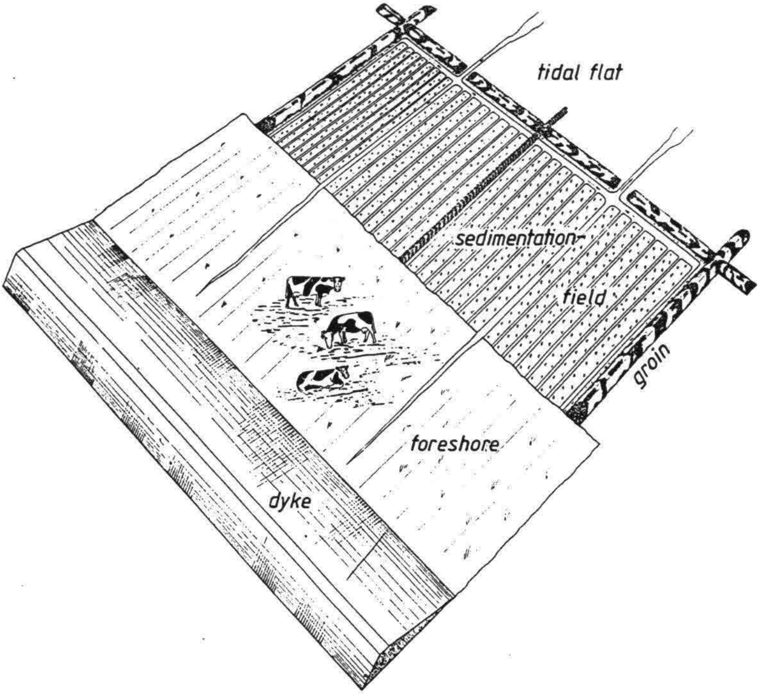


fig. 9: Dike with foreland and groin fields for active coastal protection [6].

The German Baltic Sea coast is strongly subdivided by firths, bays, glacial lagoons and barrier lagoons. It's boundary coast has a total length of 770 km which is spread over the Federal states Schleswig-Holstein and Mecklenburg-Vorpommern in equally long parts. The shallow coasts proportionally take 270 km in which dunes, dikes or dikes and dunes in combination, if necessary supplemented by a coastal protection wood, protect the hinterland from storm surges (table 2). At 70% of the boundary coast of Mecklenburg- Vorpommern the forces lead to a reduction of the coast (fig. 10). Pile groins, breakers (fig. 11) and sand nourishments are used against this.

Table 1: Lenth of the coastline and the maindikes at the German North Sea Coast

State	**lenght of the main dikes (km)**		
	mainland	**islands**	
Niedersachsen	615	133*	
Bremen	83		
Hamburg	100		
Schleswig-Holstein	292	66	
total	**1.090**	**199**	Σ **1.289**

*incl. protecting dunes (dunes with the function of dikes) on the islands of Niedersachsen

Table 2: Length of the coastline and the main dikes at the German Baltic Sea Coast

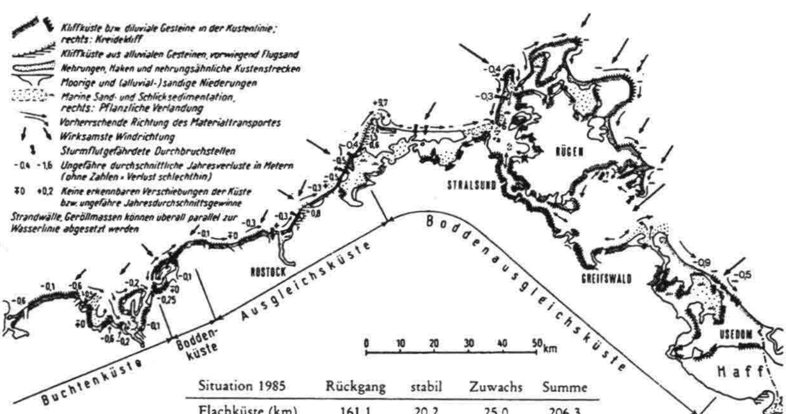
State	outer coast (km)	shallow coast with dikes and/ or dunes[*]
Schleswig-Holstein	328	68
Mecklenburg-Vorpommern	340	206
total	**668**	**274**

[*] dikes and/ or dunes, in several cases with protecting forest

3 Coastal protection management

Coastal protection is the matter of the coastal states which are the three states Niedersachsen, Schleswig-Holstein and Mecklenburg- Vorpommern as well as the two city states Hamburg and Bremen (fig. 1).

The single Federal states have arranged and published the necessary coastal protection measures in a general plan. The coastal states also arrange the fundamental plans for single measures and the legal regulations by their water engineering administration. Their authorities usually take care of the carrying out of the building measurements.



Situation 1985	Rückgang	stabil	Zuwachs	Summe
Flachküste (km)	161,1	20,2	25,0	206,3
Steilküste (km)	77,5	56,2	-	133,7
Summe (km)	238,6	76,4	25,0	340,0
Verteilung (%)	70	23	7	100

Fig. 10 Map of the Baltic Sea Coast of Mecklenburg-Vorpommern, which shows the dynamic of this coast, especially the eroding beaches („-0,5" means: average yearly erosion of 0.5 m)

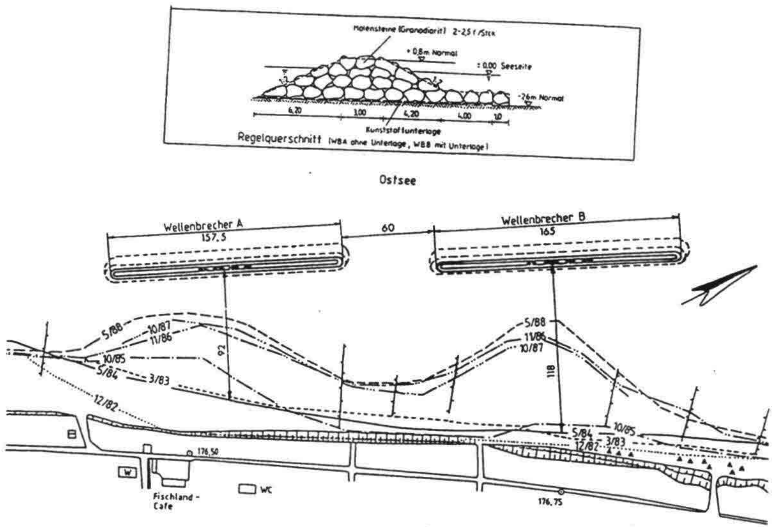


fig. 11: Change of the shore line in 1982/88 by sand deposits in the area of breakers in Wustrow/Fischland, Baltic Sea coast [9].

The expenses for creating all the protection measurements are carried by the single state with 30%. 70% of these expenses are carried by the central government.

There are different regulations concerning the maintenance of the dikes: the main dikes at the mainland coast in Niedersachsen and Bremen are mainly carried by dike associations due to a tradition that has been lasting for centuries in which property owners of the areas which are endangered by storm surges are joined together (fig. 12). That means, that all people living in areas below the contour line with the height of the highest expected stormtide water level belong to the associations. The property owners carry the expenses together either according to the scale of their share of area or their protected fixed assets. The federal states carry the expenses for the protection of the islands and the operation of the barrages (fig. 12) and also the maintenance of the dikes in Schleswig- Holstein, Hamburg and Mecklenburg-Vorpommern without a special charge for the local dike associations.

The storm flood warning service has been developed by the coastal states. The warning service tries to inform the local responsible people about the expected highest high water level 6 to 3 hours ahead of its onset (fig13).

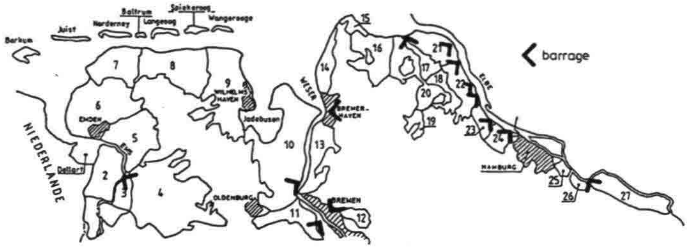


fig. 12: 27 Dike associations in Niedersachsen and barrages in the tidal rivers [8].

4 Coastal research

The important research for the principle and precaution plans in coastal engineering is mainly done at universities. An important resource for this is the large wave flume of the universities of Hanover and Braunschweig. A

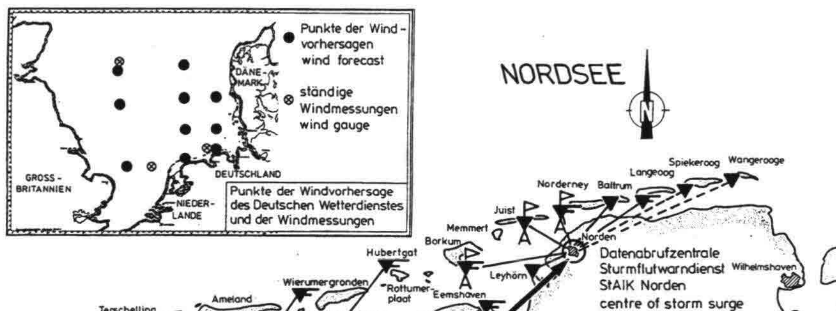


fig. 13 The storm surge warning system gets actual dates of storm surge water levels from gauges of the Netherland and East Frisian coast and wind forecast as well as actual dates from wind gauges of the North sea and ist coasts.

prerequisite for the design and construction of coastal protection structures is a study of morphological and hydrological conditions as well as dynamic loading, especially due to wave action. Besides this, the federal government and the coastal states carry out practice orientated research plans with their authorities combined in the curatorium for research in coastal engineering. The research is predominantly financed by the responsible federal minister, in individual cases also by the European Union.

Over and above that, technical- scientific corporations also support the research in Germany and they also take part in the preparation of recommendations for example for the execution of coastal protection constructions.

5 Coastal protection - protection of nature

Considerable areas of the coast are located within the local national parks or within nature reserves. The absolutely natural ecological development that is strove for in these areas is not always compatible with the tasks of coastal protection. Planned building measures of coastal protection sometimes require intensive examinations about the compatibility with the environment, the so called environmental impact assessment studies. The interventions in the ecosystem generally has to be balanced out or compensated by substitution measurements.

Dike foreland, salt meadows and dunes that are especially valuable areas in nature are kept under special protection status. Every single maintenance task in these areas needs to be matched for coastal and natural protection. For this reason, management plans for the single coastal areas are being developed lately.

6 References:

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The Scientific and Technical Development of MarineTechnology in the Central Weather Bureau

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Abstract

Forecasting, observation and data-base are the three major components of the Marine Meteorology Center that was established in 1993. The observed data and information contained in the data-base are utilized in the marine forecasting, and the forecast accuracy is relied on the integration of understanding natural phenomena and upgrading prediction techniques. Therefore, in the future, the scientific and technical development in the marine forecast will, in principle, aim at elevating the forecast ability, the accuracy of observation, and techniques of data quality control.

The main directions in upgrading forecast techniques are model development and verifications. The developments of large-scale and coast models and the study of their physical properties are the main goals, such as the regional tidal characteristics and its related tidal current, the regional wave and storm surge characteristics, and their interaction. For model verifications, it is necessary that observed data from deep sea to coastal water should be included in order to improve model performance. In the future, the remote sensing observations, encompassing the wide range image processing ability, will be cultivated to incorporate the wind fields, waves and surface currents, and also be furnished to initialize and verify ocean numerical models. For data quality control, the emphasis will be on the imminent data quality control and the research of the climatological data quality control.

1 Forward

Taiwan is surrounded on all sides by seas and oceans. With the development in economics and the increase of the population, the short falls of the increasing deficiency in land-mass resources and space exploitation. Thus this causes an increasing importance and urgency in developing and exploiting the marine resources, and in navigating safely in seas or oceans. Considering these very considerations, the Central weather Bureau (CWB) on July 1, 1993, established the Marine Meteorology Center (MMC). That is a department responsible for the marine phenomena as a first step in work of oceanic development.

Marine phenomenon is defined as the natural phenomena of the interface between the ocean and the atmosphere, such as the sea surface wind field, the temperature field, tides and waves. Therefore, there are four work targets, i.e., Observations, forecasts, data banks and operation refinements in the

MMC. All these works are accomplished stepwise; and tides, storm surges, and ocean waves are the primary work for the present. Right upon the establishment of this center, there came a personnel refinement and simplification project. The man power and the budget are largely restricted, which leads to a slow progressing pace in fulfilling the marine observation and forecast operations.

This introduction will summarize the current operational status and results accomplished in the MMC. And also it will state the problems, which are the direction of researches in the future developments, encountered in pushing the whole project through.

2 The current status of the marine and forecast operations

(1).The marine observation stations along sea shore

There are 13 tidal and five wave stations (Figure 1, 2) belonging to CWB, installed along shore area around Taiwan. All these stations are oriented in collecting the increasing or decreasing water levels along shore, and the ups-and-downs status of waves on the ocean surface. All these observed data, through telephones or digitized cables, specifically gathered up and transmitted to the CWB. Then they provided for the forecast materials of fishing, crashing smuggling, information over inrush of the sea water and safety of navigation on sea areas.

(2).The Marine Data Bank

CWB has the most abundant marine data than any other units in Taiwan, it has the responsibility setting up archives of the observed marine data that can be applied by other related units. And now the marine data inquiry and the exhibition system have already been established on the World Wide Web (WWW).

(3).The Marine Forecast

1).Tide and storm surge Forecasts

Currently, weekly tidal forecasts are issued. The forecast spots are of 24 areas (Figure 3) along sea shore around Taiwan. Information are for the mass media use, and for references for the government agencies and general public in order for the coast guard, sea shore engineering and recreational activities. With the exploitation of the numerical storm surge model, the forecast maximum water level and the occurrence time are issued within the effective forecast interval based on the typhoon warning issuance upon the invasions from typhoons around Taiwan area. These forecast data, as compared to the height along the coast bank, can appropriately in time provide surges or inrush water warning. They also play an important active role in disaster's prevention.

2).Wave Forecast

Currently, charts of OOZ wave analysis and the next-24-hour wave forecasts (Figure 4) are issued daily, with ranges covering starting from 105E to 140E, 5N to 35N, which encompassing the Taiwan Strait, the Eastern China Sea, the Yellow Sea, the Southern China Sea and a portion of the Northern Pacific Ocean. They benefit rather much for the navigation safety over sea areas around Taiwan.

3 Problem encountered

In advocating the marine observation and forecast operations, due to deficiency of man power and a limited budget, the observation stations in far-out sea areas and along-the-coast regions can not be widely set. Under this circumstance, the data collections and forecast services can not be elevated and the pre-set purposes can not be reached. While the quantity and the quality of the observed data will influence the end result of the forecasts. Besides the observed data, the forecasts also need forecast models as a necessary auxiliary in order to completely achieve the forecast targets. Thus the elevation in the forecast skills will rely on the developments in forecast models, which in turn need the observed data as an aid in understanding the characteristics of the physical phenomena, in verifying model performances and improvements. Accordingly, the problems encountered can be classified as follows:

(1).Deficiency in man power and limited in budget fees,

(2).Being unable to be widely installing the marine observation station,

(3).the marine data quality control,

(4).comprehension of the marine phenomena characteristics,

(5).Elevation in the marine phenomena forecasts.

4 Future developments

The CWB is an operational department and also a service unit for the outer world. The elevation in service counts is not only on the fruitful observed data but also on the forecast accuracy. Thus, the future aim will be focused on widely installing the observation stations, the establishment of data control, the characteristic analysis of the marine phenomena, and model development and improvements.

(1).The observing network of the marine phenomena

Incorporated with the points of view of forecasts and observations, station installments should be under taken from the environment assessments. Firstly, the historic fragmented data should be collected and manipulated in order to analyze the general characteristics of the marine phenomena. Considering this property, the stations can be established. They include the distribution of the wave stations from the deep sea area to the along the coast ones, the proper installing arrangements of tidal stations and the establishment of the station of storm surges.

(2).The marine data quality control

The data quality control can be divided into two aspects, one is the instantaneous observation data, and the other is the data quality control of the climatological data. The former is closely related to the instrument cultivation, filtering and the geological environments. The latter will depend on theories to discriminate the rules of the data quality control.

(3).The characteristics of the sea-surface wind field

Around the Taiwan island, there are three off-island meteorological observation stations (Figure 2). It is due to the non-representative of their meteorological data, such as the wind direction, the wind speed, in sea-surface wind data. Thus the observing posts, platforms, and data buoy on the sea surface are used to proceed the sea surface observations and to investigate the vertical distributions of sea surface wind profile under different stability. Afterwards, these characteristics are applied in analyzing the statistics of winds between off-island stations and sea-surface.

(4).The characteristics of tides, meteorological surge, and wave around the Taiwan sea areas

1).The characteristics of the tides along the shore

The sea shores around Taiwan island vary greatly with 3000-4000 meters deep to the east coast, a few hundred meters deep in difference between the northern and southern tips, which the west coast has a depth within 100 meters (Figure 1). Thus as the pacific astronomical tides enter the Taiwan sea areas, the characteristics of the tides will conform to a regional variability with influences from the topography of the Taiwan island. These special traits include the physical mechanisms as the tide types, the tide timings and the tide ranges, which, simultaneously, include the relevance between the along the coast tide and the ambient currents.

2).The characteristics of the meteorological surge along the Taiwan coast

During the winter, the Taiwan area is under the influence by the continental cold high pressure or southward moving front accompanied by the cold outbreaks. This will cause a water level change in the northern tip. During the summer, Taiwan area is also influenced by typhoons. The water level will also be affected. These variations in the water level variations are generally called wind-surge, which have otherwise called the meteorological surge. Thus, the different distributions and statistical properties along the coast water level variations around the Taiwan island can be analyzed under different invasion paths of typhoons first. The physical mechanisms of the water level change are simulated with three dimensional model around the Taiwan island. In the winter time, being due to the abrupt pressure fall caused by a frontal passage leading a variation in the water level will affect tides, it is a worthwhile research project.

3).The wave characteristics along the Taiwan coast.

Three main meteorological conditions, the northerly in winter, southwesterly and typhoons in summer, are the capital factors in influencing the wave characteristics around the Taiwan sea areas. During the winter, fronts moving southward accompanied by the northerly, the wave growth in Taiwan sea areas and spectrum variations are mainly discussing the regional wave characteristics. During the summer, the relevance between the swell caused by southwesterly and the sea-land wind effect is the research topic of the forecast. During the invasion or under the influence of typhoons, the wave characteristics of the specific sea areas have always been stressed by all who are interested in wave forecasts. The coming world trend is to establish a model suitable for the Taiwan sea areas in order to simulate and investigate the wave climatology.

(5).Applying the remote-sensing technique to develop the

marine numerical modes.

It's rather expensive to install observation stations on the sea surface, and indeed with a big adventure. Under such circumstances, the application of the remote-sensing technique in investigation the physical mechanism of marine phenomena is the best method, such as the freewide sea surface wind field, the sea surface spectrum and water level, etc. With its large range image manipulations, it can suffice the data inefficiency of the observed data on the sea surface, also benefits in understanding physical phenomena. The manipulated data can be supplied as the initial field for the numerical models, also for the verification uses.

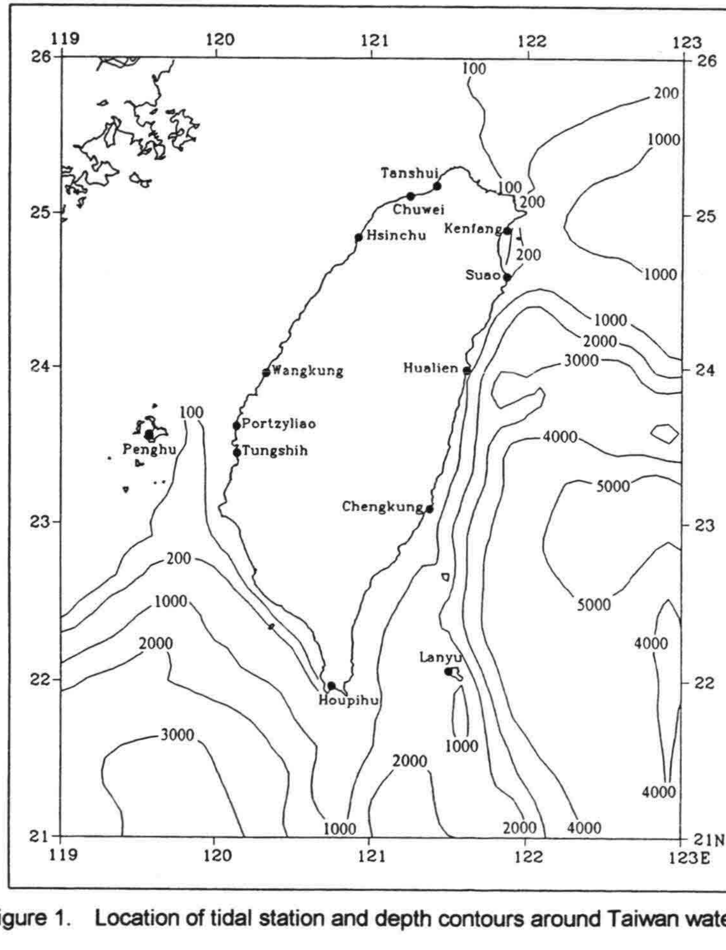

Figure 1. Location of tidal station and depth contours around Taiwan water.

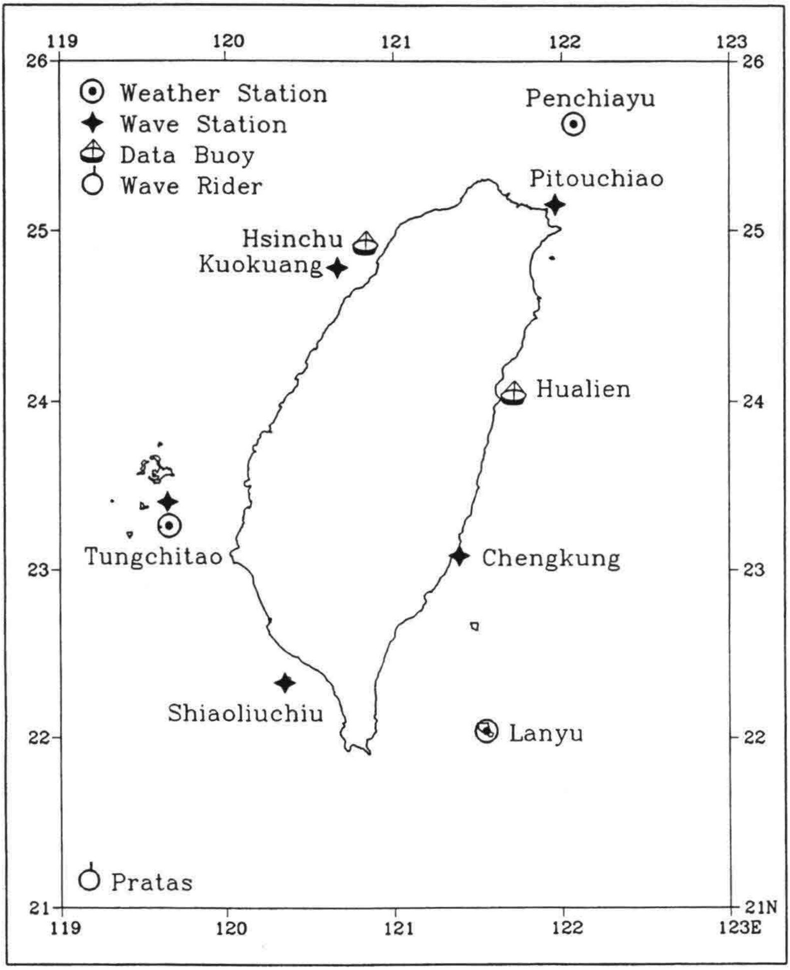

Figure 2. Location of wave and off-island stations.

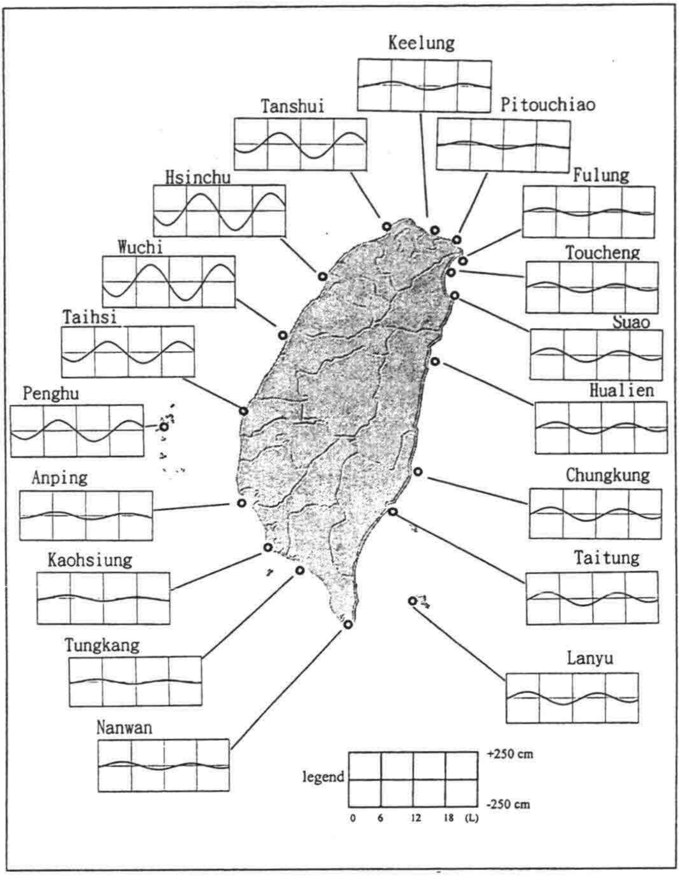

Figure 3. Sample tidal forecast chart.

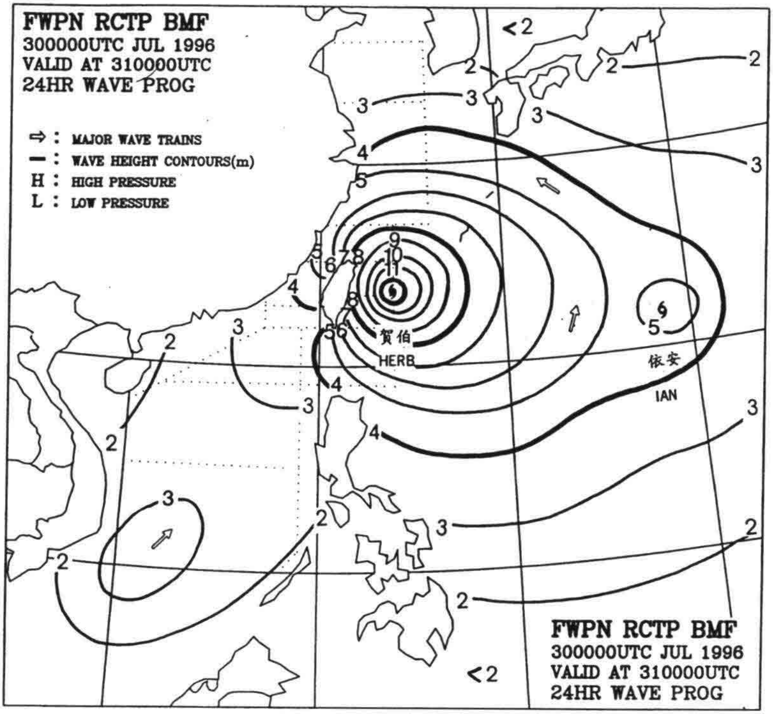

Figure 4. Sample wave forecast chart.

Climate Impact Research for the German Coast of the Baltic Sea

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Climate Impact Research attempts to analyze the influence of climate change on natural and man made systems. With regard to coastal engineering it uses the results of climate research and links them with the complex interacting processes of wind, waves, currents and morphological response. This implies that numerical models have to be available not only for the individual processes, they have also to be coupled to describe the total coupled system.

Inspite of the significant advances in the validity of numerical models a quantification of errors can seldom be made, even for the individual models. Coupled models are even much more prone to adverse error propagation. There still exists a conflict between the desire to know responses to a climate change on one side and the insufficiency of coupled models of atmosphere, sea and morphology on the other side.

The paper addresses this issue on the basis of an ongoing projekt for parts of the German Coast of the Baltic Sea, it presents the concepts chosen, and gives first results.

Topic I

Coastal Protection

Land Reclamation and High Water Protection

Chairman: Nai-Kuang Liang

Integration of Coastal Protection in Germany into a Coastal Zone Management

Hans Kunz

Abstract
The coastal areas boardering the German part of the North Sea are lowlands. Coastal protection - particularly flood defence, land reclamation and erosion control - has been carried out over centuries. The maintenance and enhancement of the coastal protection system is essential for the security of the people and the development of the coastal zone. The applied protection strategy has to be performed in accordance with the actual demands of society. Instead of land reclamation, which is no goal any longer in Germany, multifarious environmental targets have become increasingly important during the last decades. Their combination with the traditional needs of the coastal community produces the targets which specify locally the general principle of 'sustainibility', which has to be met by the applied coastal protection strategy. This modification of society necessitates a change from the traditionally sectoral approach of a 'fixed protection line' towards new strategies, which dynamically manage the coastline by integrating a flexible coastal protection strategy into the concept of Coastal Zone Management.

1 Introduction
The present shape of the southern North Sea coast is the latest transitional stage of a changing and in no way completed geological process. The German part of the North Sea coast and adjacent parts of The Netherlands (west) and Denmark (north) are drawn on Fig. 1. The displayed classification is based on tidal range (Hayes, 1975). The East Frisian islands are sandy barrier islands (dune islands) formed by the coincidence of tides, currents, surf and wind-born accretion; the North Frisian islands are the remainder of former mainland. In the low macrotidal area there are open tidal flats with high sand banks as well as bays of estuarine flats. Sheltered tidal flats extend behind the islands and salt marshes spread behind them as a more or less extended transitional zone. The mainland is low lying marshland with intercalated peat layers and areas of peat bogs. Especially along the estuaries the lowlands extend far into the mainland.

The recent history of the people living in the coastal areas of the Southern North Sea is mainly a history of retreat: Reliable data give the information, that the coast line had been located about 250 to 300 km further seawards nine thousand years ago (Behre, 1987). Coastal protection and water management is part of man's political, social, economical and technical history throughout the last thousand years. The effort of coastal protection has always been based on

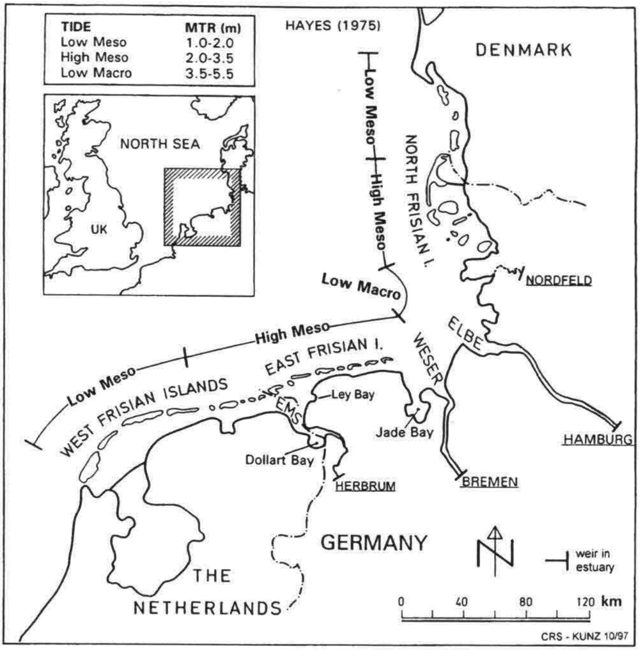

Fig. 1 Location map of the German North Sea coast with parts of The Netherlands and Denmark

managerial structures, which were altered dynamically by responding to the respective problems, objectives and means. The responses included a variety of aspects associated with the coastal protection setpoints, e.g. drainage, shipping, farming. To be able to make structurized analyses of the management processes, a simple 'model of thinking', as show on Fig. 2, may be used. The basic thought of the model is, that coastal protection, as well as integrated aspects, are determined by „key factors": physical conditions, social surroundings, social needs (asking 'why?') and social factors: options, setpoints (asking 'how?'). The four basic key factors determine by their mutual coherence and relations how developments perform (Kunz 1993a).

The protection against storm floods (flooding) and erosion (land losses) is essential. Concerning coastal protection technique we can state: an appropriate technique is available to adapt the established defence line along the German North Sea coast to a relative sea level rise of 1.0 m or even more. The special problems for the harbours and low lying settlements as well as those on the islands could be dealt with; hence it would be possible to maintain the existing coastline and coastal defence line ('main dike-system') along the German North Sea coast without retreat (e. g. Kramer & Rohde 1992). Concerning nature conservation we have to be aware of, that the Wadden Sea and the forelands along the estuaries have been widely shaped by mans activities throughout the last

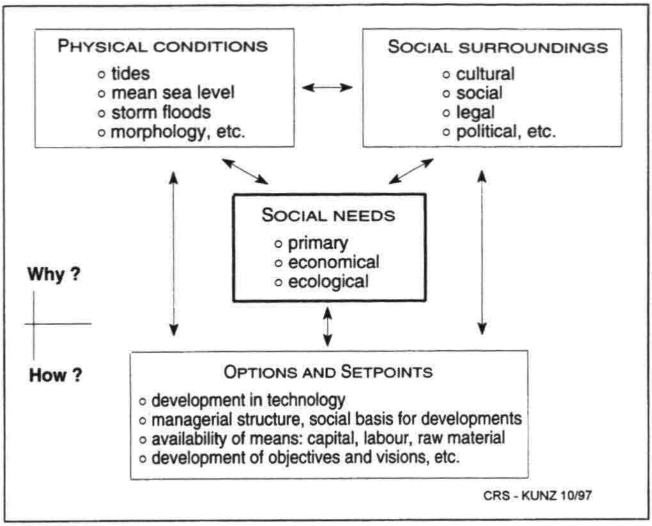

Fig. 2 Scheme of combined 'key factors' related to coastal protection-management

centuries. However, the targets of nature conservation and National Parc interfere with the mandatory protection strategy. This is especially the case in areas with predominantly erosive trends and small distances between the coastal protection line and the sea, when retreat is demanded, instead of fixing the coastline.

The contradictionary aims can be adapted to each other by compromises only within a limited range, hence decisions on priorities are needed. This leads to thoughts about conversion of the actual sectoral orientated coastal protection strategy towards management, which mutually integrates other demands of the coastal society (e. g. Kunz 1993b).

2 Coastal Management Strategies

The development of coastal protection management in Germany towards a less sectorally oriented strategy shall be demonstrated by the example of the Ley Bay, East Frisia. Refering to Fig. 2, the description restricts on the major facts:
Physical conditions: The Ley Bay attained is largest extension of about 130 km²
after the disastrous storm floods of 1374 and 1377. Afterwards this bay was far too large with respect to the hydrodynamical boundary conditions. Hence silting-up took place, accelerated by land reclamation and diking works (Fig. 3). The open bay-area after the closure dike of 1950 (Fig. 4, top/left) still had not reached equilibrium conditions.
Social surroundings, social needs: The diking activities associated with flood defence, land reclamation, drainage, shipping were based on the respective

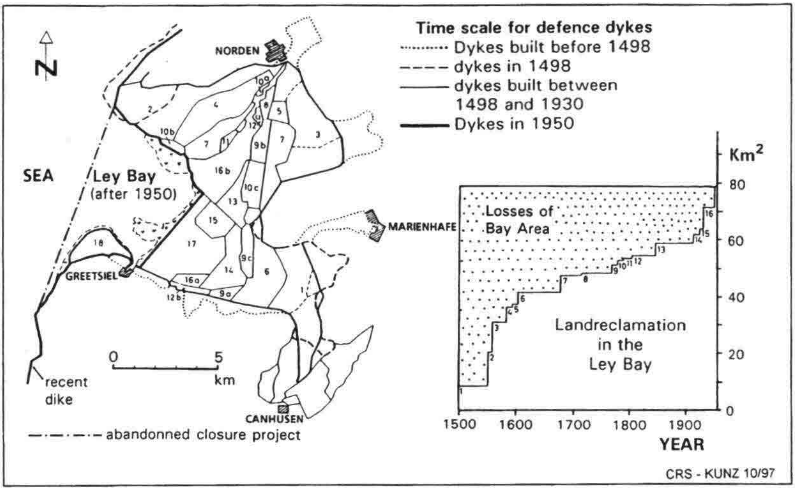

Fig. 3 Time history of the Ley Bay reclamation since the 16th century; left: location of dikes; right: areas of polders (adapted from HOMEIER 1969)

cultural, social, legal and political conditions. Different interests of the coastal society hat to be balanced. Needs for measures arose after the 1950 closure again, mainly because of sedimentation affecting drainage and navigation (fisher- and pleasure boats) and of the fact, that the existing dikes did not meet the new safety requirements developed after the 1962 storm flood in Germany. The coastal society followed the traditional approach: social needs refer only to the primary and economical factors, without taking ecological demands substantially into account.

Options and Setpoints: Technique, experience, managerial structures and means had been present for a total closure of the Ley Bay. Hence, there were no really restrictions deriving from the 'option and setpoint - key factor', as it has been the case in the past. The new plan of 1961 for the Ley Bay-area (Fig. 4, top/right) had been based on the traditional coastal protection - principles, prospecting the ultimate enclosure of the Ley Bay. This plan became increasingly subject to a controversial discussion focusing on necessity, on economics and on ecological aspects (Hartung, 1983). This discussion widened the narrow sectoral approach and introduced several requirements which had to be met by the project simultaneously:

- Sea defence according to the valid Lower Saxonian safety standards.
- Conservation and development of the Ley Bay as an unique ecological entity by minimisation of impacts.
- Sufficient inland drainage, mainly by gravity discharge.
- Good access for the local fishing vessels to the harbour of Greetsiel.
- Preservation of the harbour and the typical village character of Greetsiel.

Numerous alternatives had been discussed (Fig. 4, bottom/left). The final plan (Fig. 4, bottom/right) was a compromise: Construction of a peninsular-polder with discharge sluice and navigation lock which incorporates the drainage and

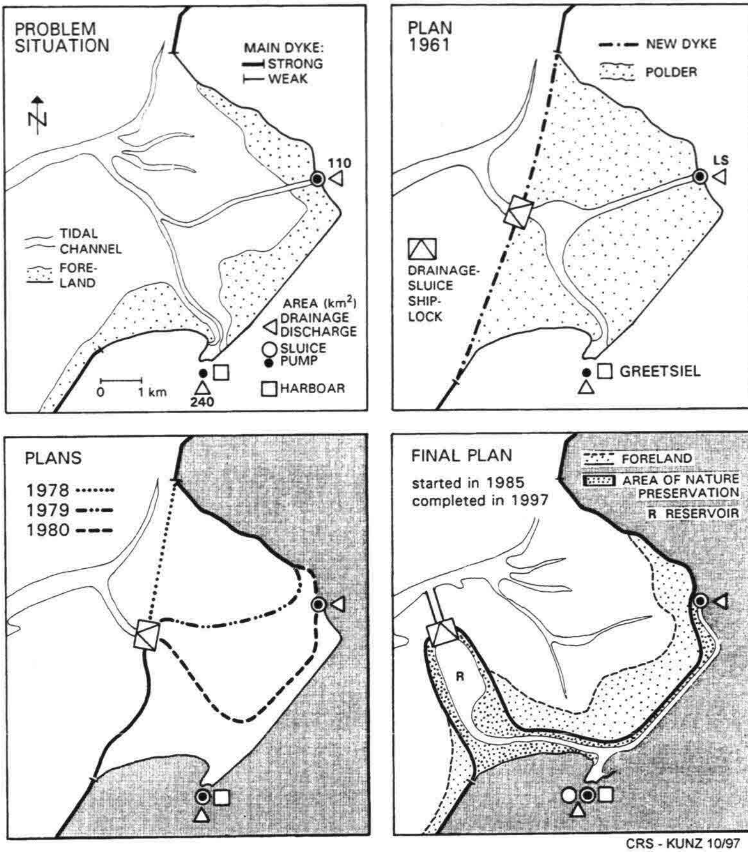

Fig. 4 Development of the Ley Bay coastal protection-plans from 1961 to 1985

navigation links to the indicated pumping stations, sluices and harbour. With respect to the high ecological quality of the project area, measures for ecological compensation and translocation (i.e. opening of a summer dike, intake of salt water, nature preservation area for birds, restrictions for access) have been implemented. The work was started in October 1985 and is almost completed now.

The legal basis for coastal protection projects in Germany are the 'Master Plans' of the 'Länder' (states). They focus on the defence/protection-sector with respect to 'floodings' (security) and 'erosion' (losses of land); however, the newest plan (MBLU Mecklenburg-Vorpommern 1997) incorporates objectives of management-strategies and ecological needs. Restrictions for the coastal

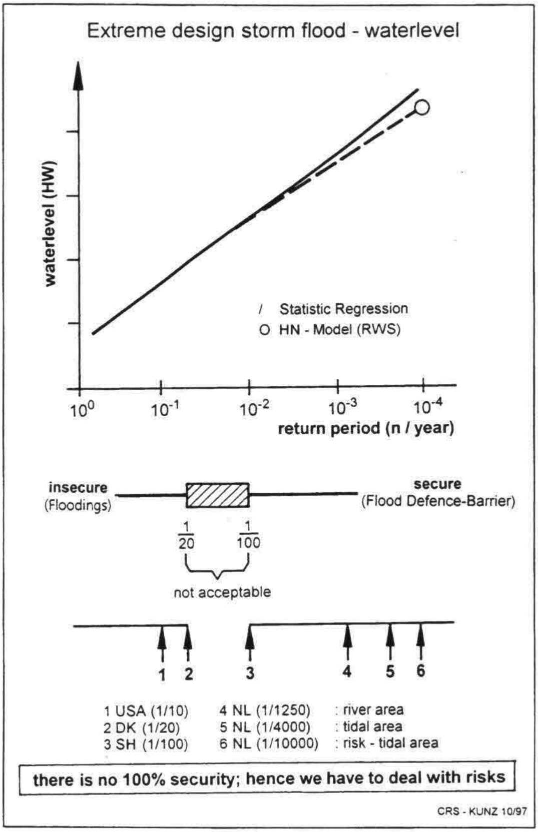

Fig. 5 Design waterlevel, return-period, security and safety-risks

protection-master plans arise from the 'Zoning Plan for the National Parc Wadden Sea' and from the 'Trilateral Wadden Sea Plan' (development from sectoral to an integrated approach).
A comparision of Germany with neighbouring European countries with respect to Coastal Management Strategies gives, in general, the following overview (e.g. RWS 1990, MAFF 1995):

Germany : Master Plans - '**Protection**'
Netherlands : Basal Coastline ('Basiskustlijn', BKL), Shoreline Management - '**Protection**'
Denmark : Flexible Management: '**Protection' to 'Managed Retreat**' (with 'set back line', Nature Preservation Act 1917, 1937)
Great Britain : Shoreline-Management Plan and CZM-Plan: '**Protection' to 'Managed Retreat'**

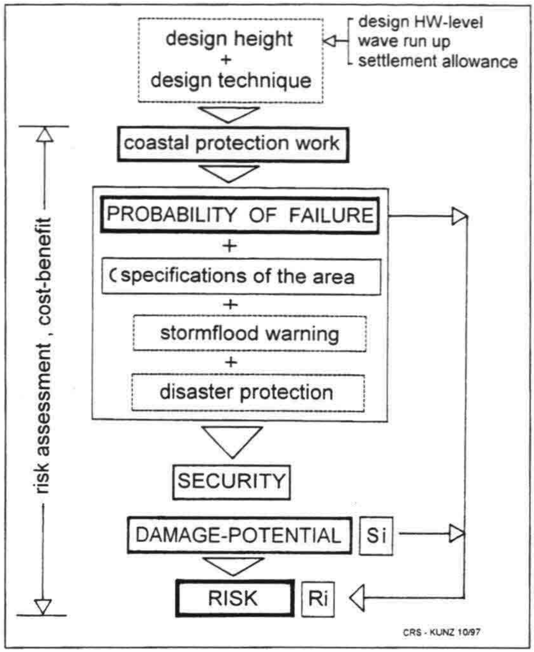

Fig. 6 Coastal protection strategy in Germany, with related 'failure propability', 'damage potential' and 'risk'

3 Process of Integrated Coastal Zone Management

Integrated Coastal Zone Management (ICZM) is an unitary program, with the goal to „manage“ the „development“ and the „conservation of natural resources“. It has to integrate the concerns of all relevant sectors of society. Essential requirements for the development of the, coastal zone are associated with coastal protection:

- Safety: protection against floodings (storm flood-hazards).
- Stability of the coastline: protection against land losses by erosion (normal and extreme events).

The priority of 'safety first', as well as the measures against erosion, have to be balanced with the guiding principle of the Wadden Sea policy, to achieve - as far as possible - a natural and sustainable ecosystem in which processes proceed in an undisturbed way. It is necessary to include the aspects of 'risk assessment' and 'cost-benefit-ratio'. This leads to the process of ICZM with a related program.

An ICZM-program has to deal with regional issues within a national framework. Each program will be unique, but has similar stages in the generation:

1. Policy formulation as basis.

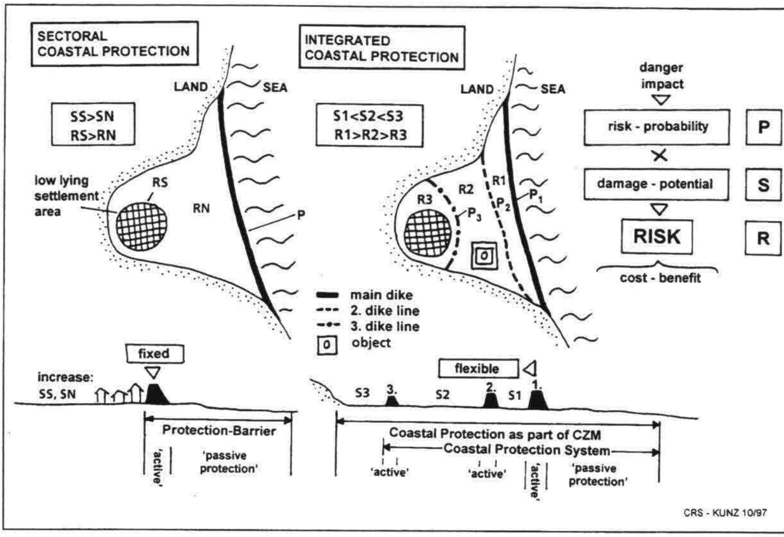

Fig. 7: Concepts for the 'coastline defence strategy' and for 'safety against storm floods'. Left: traditional 'sectoral coastal protection'. Right: flexible, integrated protection-system

2. Strategy planning, leading to a locally and politically accepted 'Strategy Plan'.
3. Program development as an approved 'Master Plan' with budjet and anthorized staff.
4. Implementation of the Master Plan.

There are international agreements (e.g. Agenda 21 of the Rio Conference 1992, World Coast Conference 1993) on the principle of 'sustainibility', which has to be implemented into ICZM-plans.

4 Safety and Risk Assessment

The extreme storm-flood water level is a basic design parameter for the protection-system against flooding. Fig. 5 shows the relation between design water level and return period in principle. The risk of flooding correlates with the return period; the lower part of the figure provides information on the return periods, which are used in different countries. Since we don't know the highest storm-flood level, we have to be aware of the fact, that there is no 100 % security achievable; hence we have to deal with risks. Risk is the result of 'probability of failure' times 'damage potential'. Fig. 6 displays for the German situation the factors which determine the failure of a coastal protection work and the risk within the protected lowlands.

The coastal protection system shall ensure, that no lives are on risk. According

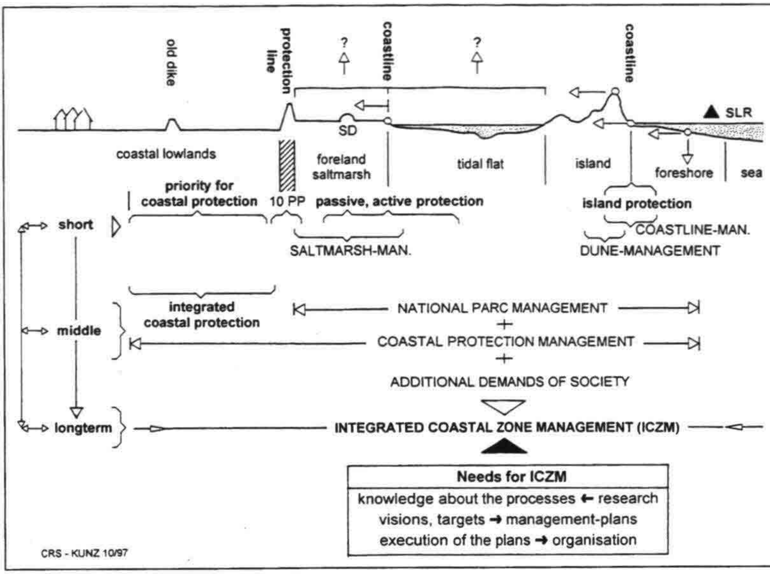

Fig. 8 Cross section for a coast with barrier islands (East Frisia, Germany) and steps for the development towards an Integrated Coastal Zone Management (ICZM) which includes Coastal Protection

to legal plans, a fixed main flood defence-line ('protection-barrier') covers the entire German North Sea coast ('sectoral' coastal protection strategy). It consists of dikes/embankments, walls (in developed areas such as settlements, industrial regions, harbours), storm surge barriers (mainly in tributaries of the estuaries), weir (artificial end of the estuary) and it contents road-gates (access), tide gates (sluices), pumps (drainage, irrigation), navigation locks (shipping). Since 1962 the length of this main flood defence line has been cut down substantially. Fig. 7 shows the situation for a protected low land area, in principal. The left part stands for the actual situation: The risk-probability (P) relates only to one defence line, which position shall not be moved in the landward direction (fixed line). The right part of Fig. 7 displays an 'integrated' coastal protection strategy, which provides flexible responses, as well concerning the risk probability of the 'active' parts (P1, P2, P3), as with respect of their position (1, 2, 3). This system allows to decrease the risks according to the damage-potential of distinguished areas. The 'integrated' strategy could include old 'second' dikelines and it would preserve options for 'managed retreat' which still exist nowadays (Kunz 1995 , Probst 1994). It is a robust solution which provides higher security standards and which and can be favourable with respect to 'cost-benefit' aspects.

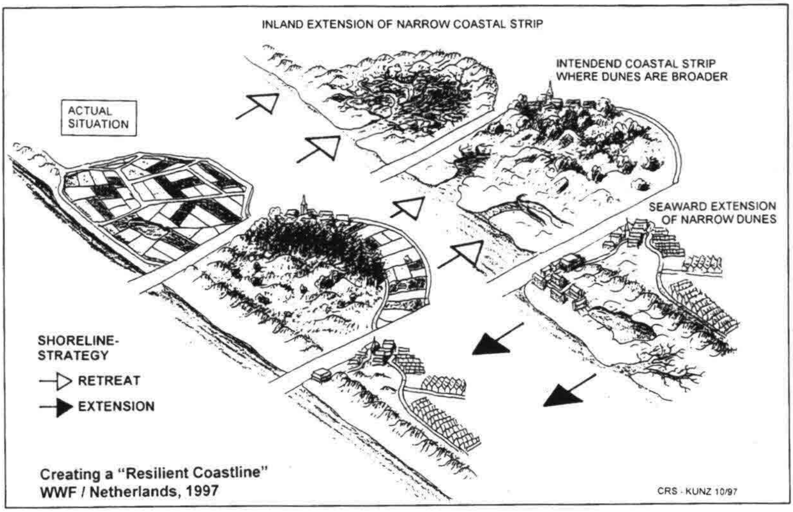

Fig. 9 Creating a 'resilient' coastline - adapted from WWF 1997

5 Integration of Coastal Protection into a Coastal Zone Management

The conversion of the existing sectoral coastal protection strategy into an integrated approach for the German North Sea coast can only be achieved step by step (short to longterm). This is principally visualized on Fig. 8. The upper part shows a cross-section for the East-Frisian situation: the sea is located to the right, the lowland to the left; inbetween are located: barrier-island, tidal flat, saltmarsh; 'active' protection stands for constructions which have to withstand the attacking forces of the sea, 'passive' protection relates to means which decrease the attacking sea-forces; SD is summer dike, 10PP stands for a special 'ten point catalogue' which addresses conflict areas between coastal and environmental protection in Lower Saxony. ICZM offers possibilities for dynamic approaches to the impacts of the sea, including responses to rising sea level in the long run. Dynamic approaches lead to flexible management strategies which comprise 'extention' as well as 'retreat' by artificially maintaining the coastline (concept of 'resilience', which stands for the ability to deal flexibly with both natural and social dynamics in the coastal area - WWF 1997). Fig. 9 pictures the concept schematically.

6 Research-projects related to Coastal Protection Management of the Coastal Research Station, Norderney (CRS).

The Coastal Research Station of Lower Saxony (CRS) deals with applied research. Much of the work is carried out in the framework of the 'Kuratorium für Forschung im Küsteningenieurwesen (KFKI)', an association of federal and state institutions, which have practical engineering duties in the coastal areas.

Coastal Protection Management: BMBF/KFKI - RESEARCH PROJECTS of CRS

(1) Dike Design due to wave impacts (BMBF/KFKI)
- Wave damping due to saltmarshes and summer dikes
- Dike geometry design
- Evaluation of wave run-up under consideration of dike geometry and overtopping volumes
- Overtopping security with respect to soil characteristics, recommendations

(2) WADE: Wadden Sea morphodynamical development with respect to accelerated sea-level rise (BMBF/KFKI)
- Parametrization of morphodynamic processes
- Evaluation of morphodynamical equilibrium conditions
- Application and development of conceptual morphodynamical models

(3) Sediment-distributionin wadden areas as indicator for morphodynamical processes (BMBF/KFKI)
- Evaluation of sediment distributions in the Wadden Sea
- Evaluation of changes over time
- Conclusions with respect to effects on coastal erosion

(4) Morphological developments in foreshore areas (BMBF/KFKI)
- Evaluation of longterm changes (steepening of foreshore areas), based on bathymetric data (German North Sea coast) and GIS-processing
- Evualation of sand transport-rates (erosion, accretion)
- Conclusions for shoreline-management

(5) NOURTEC: Innovative Nourishment Technology (EU, BMBF/KFKI)
Partners: Denmark, Germany, The Netherlands
- Evaluation of governing morphodynamical processes
- Effectiveness of combined beach and shoreface nourishment
- Recommendations for application

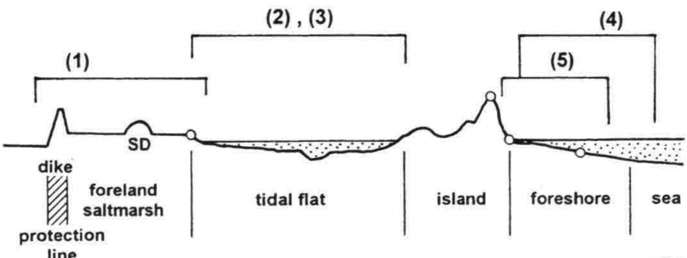

Fig. 10 Research programs of Coastal Research Station (CRS), Norderney dealing with engineering problems of Coastal Protection and Coastal Zone Management

Most of the actual KFKI-projects of the CRS are linked to 'Coastal Protection' and 'Coastal Zone Management'. Fig. 10 provides a list of these projects.

Summary
The coast of the southern North Sea has been shaped by holocene sea level rise in coincidence with storm surges. The mainland of the German part consists of low lying marsh and peat-areas; several bays have been cut into the mainland by storm floods in the past. Afterwards silting-up took place as a naturale process, supported by human land reclamation means. Parts of the bays were repeatedly poldered and additionally protected by embankments. This has been based on the agreement of the coastal community that land reclamation and coastal defence is essential. Since some decades the targets of nature conservation have become increasingly relevant. They interfere with the established coastal protection strategy: the latter favours to defend the existing coast- line, while nature conservation principally promotes retreat options. However, sidespecific compromises can be achieved, especially since land reclamation is no goal any longer. The actual discussion in Germany on compromises includes the demand to decide politically on priorities with respect to the necessities of coastal protection and flood-defence on the one hand and the demands of environmental protection on the other hand. This problem can only be analysed and validated against the backdrop of the polical, social, economical and technical history. Thus, the effort of the coastal people in Germany protecting their lives and property by sea defence means has always been based on managerial structures. The procedure of the coastal society to decide on concepts and to implement techniques were altered dynamically over time and can be addressed in principle, as Coastal Zone Management. The management decisions on social needs - safety, economy, ecology - are determined by 'key factors' asking 'why?' and asking 'how?'. Looking at the North Sea-area, we can recognize, that the development of coastal protection performed differently, because of the sidespecific conditions in every country. However, refering to 'sustainibility' and 'resilience' all countries nowadays agree in principle, that coastal protection has to relate to the internationally reconciled principles of Integrated Coastal Zone Management. Comparing with the neighbouring countries, we have to acknowledge, that coastal protection in Germany is still mainly based on sectorally orientated decisions, which not really take into account the aspects of risk - analysis, cost-benefit-ratio, probabilistic approaches and sustainibility. A good integration of these aspects into a Coastal Zone Management framework within a long term scope is favourable to ensure that the existing high standard of coastal protection along the German North Sea coast can be maintained and the problems to come can be dealed with in a safe and flexible way.

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The Ocean Space Utilization and Monitoring Program in the Western Coast of Taiwan

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Abstract

The development and utilization of ocean space lead to the environmental impacts. Although the environmental impact has been assessed before construction, the real impacts are monitored during the project construction and after implementation operation respectively. In recent years, two biggest reclamation areas have been carrying out in the western coast of Taiwan. To prevent the utilization of coastal area from human-caused environmental disasters, the related researches are conducted before construction and the monitoring program is set up under construction and after operation to evaluate the environmental impacts.

1 Introduction

Taiwan is an island of 36,000 Km^2 and has 1140 Km coastline. Its east coast faces the steep Pacific Ocean, however, its west coast connects with Taiwan Strait. Most of economic activities developed in the western part of Taiwan, the major coastal engineering including harbor structures, ocean outfalls, power plants and industrial parks are, therefore, constructed along the western coastline as shown in Fig. 1. Four international harbors, three domestic commercial harbors and about two hundred fishery harbors have been constructed around Taiwan island. Wherein, most of the harbors are located in the western coast. To take advantage of cooling water, all the thermal and nuclear power plants are built nearby the coastal area. Furthermore, three ocean outfalls have been operated to discharge the industrial waste water and domestic sewage separately in the vicinity of Kaohisung coast, and the biggest ocean outfall of the island has just been completed in the northern coastal area. Monitoring programs have also been set up for the above mentioned outfalls to better understand the possible ecological and environmental impacts.

In addition, in order to obtain new industrial lands and to control pollution problems efficiently, the government decided, since 1990, to develop the offshore industrial areas in the deposited western coast. Since these areas require a great deal of construction materials, among them, sea sand will be drawn from offshore to fill the reclaimed land. It is certainly expected that the reclamation influence on the ocean environment, inland discharge, pollution dispersion and so forth could be limited to a certain extent. Therefore, the related researches have to be completed before construction, and the monitoring systems would also be set up around the developed areas.

2 The Establishment of Background Information

Data collection with analysis and field investigations on the developed coastal area are the most decisive issues for the ocean engineering. Fig. 2 shows the distribution of the Yun-Lin Offshore industrial reclamation area in the western coast of Taiwan. This area, located on the south-western coast, was planned in 1992 to be built up as the biggest offshore industrial park, about 15,000 hectares, in Taiwan, as shown in Fig. 3.

Before Yun-Lin offshore industrial project was released by Taiwan government, data collection and relative analysis had been completed for feasibility study and preliminary planning. In order to establish the complete background information in the region of Yun-Lin reclamation area, Tainan Hydraulics Laboratory installed 20 measuring sections between 60 km alongshore, as shown in Fig. 3, from Cho-Shui River in the north to Pei-Kang River in the south. The systematical studies including oceanography, meteorology and hydrology were carried out from 1991 through 1992. The measurements consisted of the investigations of bathymetry, current characteristics, wave climate, water quality, ocean ecology, sediment transport and the others related investigations. The total investigation items and correlative research topics before construction, from 1991 to 1992, and under construction, from 1993 to 1997, are listed in Table 1.

Table 1. Field investigations and related researches on Yun-Lin offshore industrial project

Research Items		1991	1992	1993	1994	1995	1996	1997
1	Wave climate			△	△	△	△	△
2	Current measurements	△	△	△	△	△	△	△
3	Floating drogue	△	△	△	△	△	△	△
4	Estuary surveys		△					
5	Bed load materials	△	△		△	△	△	△
6	Suspension beds	△	△	△	△	△		
7	Near shore currents	△	△		△	△	△	△
8	Coastal flying sands			△	△	△		△
9	Suspension profiles	△	△		△	△	△	△
10	S.T.C. profiles	△	△		△	△	△	△
11	Coastal water quality		△		△	△	△	△

12	Ocean ecology			△	△	△	△	△
13	Ground water and water quality		△		△	△	△	△
14	Regional water discharge survey			△				
15	Tide observation station				△			
16	Offshore platform installation		△				△	
17	Geological investigations							△
18	Estuary water quality simulation			△	○	△	△	△
19	Coastal water quality simulation		△		○	△	△	△
20	Simulation of coast and river changes		△	△	△	△	△	△
21	Regional water discharge analysis			△	△			
22	Database and GIS establishment			△	△	△	△	△

3 Monitoring Program

According to the regulations of Environment Protection Administration of Taiwan, the complete monitoring program of coastal space utilization project has to be proposed in the environmental impact assessment. Every monitoring report must be submitted to EPA per season, and the field trip of environmental inspection should be held once half a year during the project construction. The monitoring items include air quality, noise, vibration, traffic transport, inland ecology, ground water quality, inland water quality, estuary water quality, coastal water quality, ocean ecology, fishery economy, topography and bathymetry, ocean climate and so forth. Herein, the detailed monitoring contents of this project are described as follows:

(a) Air quality: Total dust, TSP, PM_{10}, SO_2, NO_2, CO, O_3, THC and NMHC combining with wind velocity and direction for continuous twenty-four-hour detection per season. Eight observatory stations are distributed around the area.

(b) Noise: L_{eq}, L_x and L_{max} are detected every hour, then L_m (05:00~07:00 a.m.) L_d (07:00 a.m.~20:00 p.m.), L_e (20:00~22:00 pm), L_n (00:00~05:00 a.m. and 22:00 ~24:00 p.m.) can be calculated. Twelve observatory stations are set around the project area.

(c) Vibration: L_{eq}, L_x, L_{max}, L_d, L_n are detected hourly during a day and continuous twenty-four hours per season. Twelve observatory stations are set around the project area.

(d) Traffic Transport: Total numbers of cars including trucks, automobiles and motorcycles are recorded by video camera during the continuous twenty-four hours. The observatory stations and detected frequency are coincided with noise and vibration measurements.

(e) Inland Ecology: Animal, plant and estuary ecology are investigated within 15km radius of constructed area.

(f) Ground Water Quality: 4 observation wells of shallow aquifer and 2 wells of deep aquifer are set around the constructed area to detect temperature, PH, EC, TDS, NTU, Cl^-, SO_4^{-2}, F^-, TOC, C_aCO_3 and heavy metals per season.

(g) Inland Water Quality: 10 sampling stations are distributed in 5 inland principal rivers and regional discharge channels around this area. 33 items of water quality and heavy metals are investigated per season.

(h) Estuary Water Quality: 20 sampling stations are distributed in the estuary along the coastline. 24 items of water quality and heavy metals are investigated per season and after each storm.

(i) Coastal Water Quality: 6 sampling sections are installed in the coastal area as shown in Fig. 4. at -5m, -10m, -15m, -20m and -30m water depth, the samplings are taken from water surface and bottom respectively per season.

(j) Ocean Ecology: 7 sampling sections are installed in the coastal area as shown in Fig 5. Phyto-plankton, zoo-plankton and benthos are investigated per season.

(k) Fishery Economy: Fishery products including in-shore-fisheries, deep-sea fisheries and aqua-culture are investigated year by year.

(l) Topography and Bathymetry: The bathymetry surveys between 60km alongshore from Cho-shui river in the north to Pei-Kang river in the south, and the detecting water depth reaches to -35m.

(m) Ocean Climates: Tides, currents, waves and winds are investigated from the platform which is constructed at -15m water depth. In addition, five current measuring stations and one wave measuring station are installed in the coastal area per season.

It is worth pointing out that the concentration of suspension solid is increasing during the drawing of sea sands from the offshore to fill the reclaimed land. In order to monitor the concentration of suspension solid and the corresponding effect on ocean ecology, the diffusive area is studied by numerical simulation for field survey. One of the examples is shown in Fig. 6.

4 Conclusion and suggestions

The western coast of Taiwan is an over-utilized coastal region in which the Yun-Lin offshore industrial area is the biggest filling land engineering. No doubt, this engineering would lead to impacts not only in coastal environments but also in ecology. The complete database of the background information have to be established for environmental impact assessment and engineering planning and design. Although the related researches and impact evaluation have been made in this stage, the real and detailed influences on environments and ecology are still not accurately and completely known. It is the reason why the long term monitoring system will have to be set up around the developed area and further analysis of impact assessment obtained from the comparison between initial database and monitoring results needs to be continued during the project construction and even after implementation operation.

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Fig 1 The Major Coastal Engineering Around Taiwan Coast

Fig 2 Distribution Chart of the West Coast of Taiwan Reclamation Area

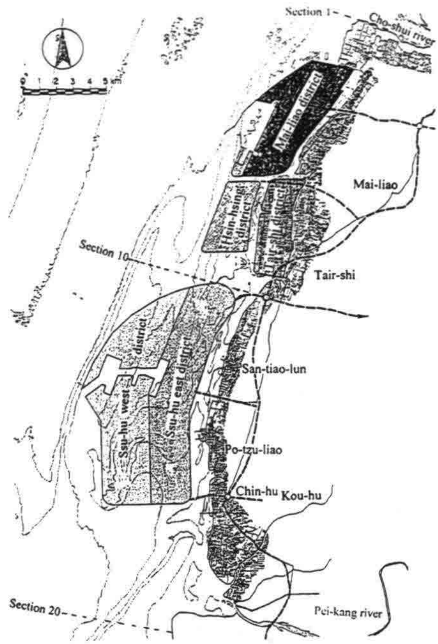

Fig 3 Distribution Chart of Yun-Lin Offshore Industrial Area

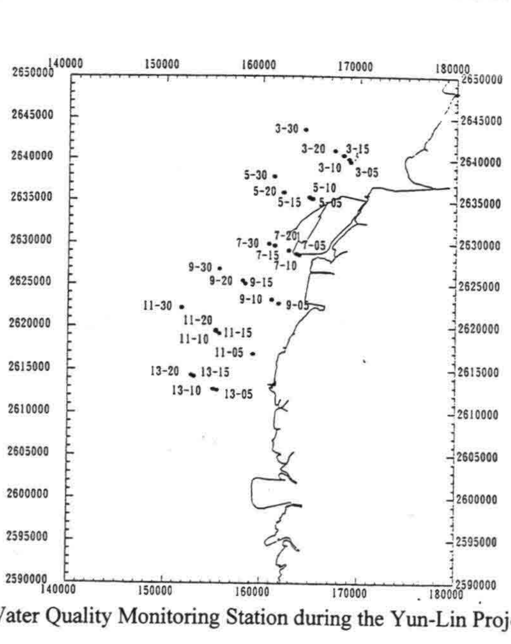

Fig 4 Coast Water Quality Monitoring Station during the Yun-Lin Project Construction

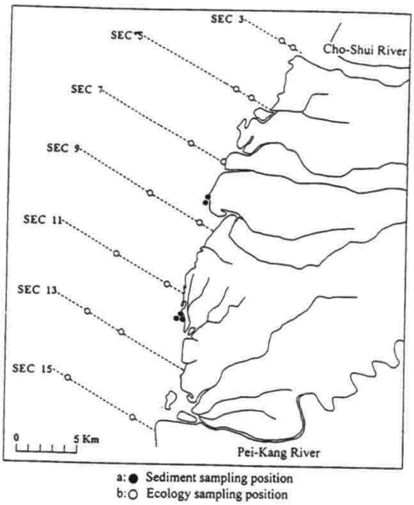

Fig 5 The Ocean Ecology Monitoring Diagram

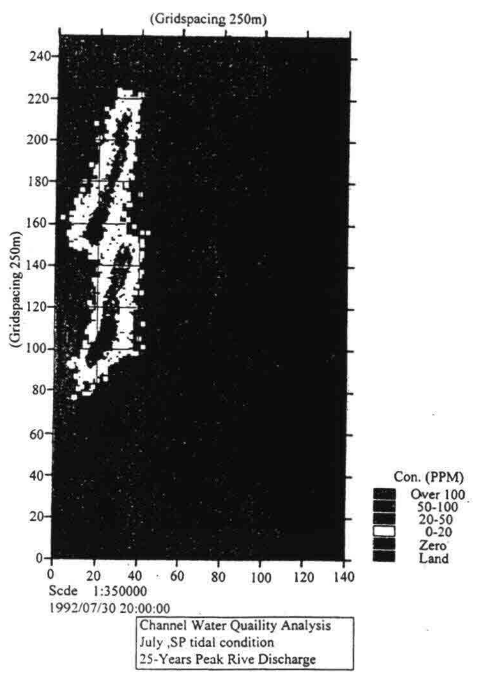

Fig 6 Diffusion of Suspension Concentration

Optimised Design of Land Reclamation Fields for Sustainable Foreland Development

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Abstract

At the German North Sea Coast forelands and salt marshes in front of the sea dikes contribute significantly to the protection and safety of the artificial coastline. In this way the forelands are an important element in the whole coastal protection system. The current state of scientific knowledge about the management of forelands is essentially based on experience acquired over generations. Therefore there is a need for research to determine the influence of the various parameters effecting accretion or erosion of forelands considering ecological aspects. Within a research programme on the optimisation of foreland management, which is sponsored by the German Federal Ministry of Education, Science, Research, and Technology (BMBF), field measurements, physical, and numerical simulations have been carried out to analyse the interaction of waves, currents, sedimentation processes, as well as design and maintenance techniques of foreland reclamation fields at the German North Sea Coast.

1 Introduction

Salt marshes and their forelying mud flats are formed by the deposition of fine silts and sands in sheltered locations and colonised by specialized salt tolerant plants (see Fig. 1). The upper mud sediment layers of the tidal flats are very active containing biological production. Very quickly vegetation starts with single plants of the three main species as follows: For the settlement of glasswort or marsh samphire (*Salicornia herbacea L.*) the terrain must be of a certain height; at about mean tidal high water level (MHW) minus 40 cm or 50 cm. This plant is annual and its height does not exceed 20 cm. In october plants die off and seeds are released. The wooden parts of the stem often remains in place as a bare bush. Glasswort does not only promote sedimentation; according to Kamps (1962) its presence can also stir up the material already deposited as a result of the remaining dead bushes causing turbulence. The second pioneering plant is cordgrass (*Spartina Townsendii*), a coarse crop-like reed (*Phragmites communis L.*), which can reach a height of about 1 m. It develops at about 30 cm below MHW and its growing period starts later than that of glasswort. The third pioneering species is sea poa (*Puccinellia maritima Parl.*). It is a grass which settles above mean tidal high water level (Schulz and Zimmermann, 1994).

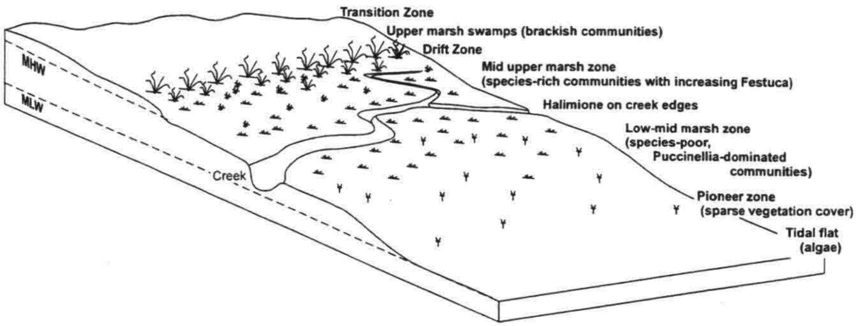


Fig. 1: Saltmarsh with Main Vegetation Zones at Tidal Levels

(after: MAFF, 1993)

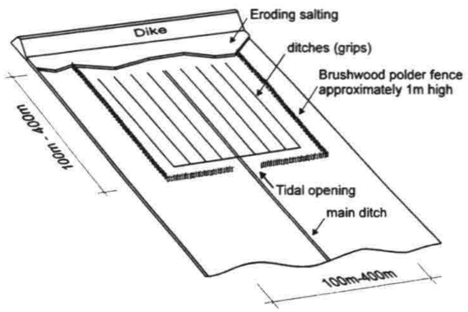
These plants attenuate tidal currents and waves resulting in increased siltation. As these young marshes slowly raise in height the tidal currents form a network of drainage channels which act as transport routes for sediments and dissipators for the tidal currents.

Sea level rise and the increased frequency and intensity of stormtides may endanger the forelands and salt marshes resulting in losses of sediment or reduction thereby decreasing wave attenuation and thus increasing erosion.

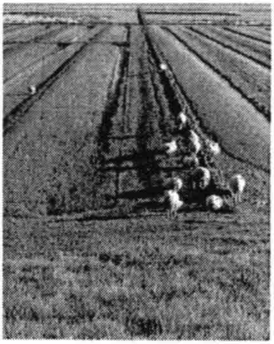
2 Foreland Stabilisation by Artifical Sedimentation Fields

In order to counteract loss of sediment and to increase natural sedimentation artificial reclamation methods have been used for generations. At the German North Sea Coast this is by systematic installation of large-scale sedimentation fields using low brushwood fences in combination with a regular drainage system (see Fig. 2). Wooden stake and brushwood structures help to create areas with decreased wave heights and reduced currents resulting in enhanced sedimentation. However, there exist limitations from local boundary conditions where such techniques are less efficient with the need for increased and ongoing maintenance.

Supporting natural and artificial accumulation of the mud-flats with support of engineering techniques increase the potential for protection of the dikes and thus ensure the stability of the coastline against waves and storm tides due to shallow water effects on the tidal flats from shoaling or breaking of waves.



a



b

Fig. 2: Sedimentation Fields with Artificial Drainage System (a) and

Traditional Land Use after Several Decades of Continuing Maintenance (b)

The dimensions of the sedimentation fields, 100 m to 400 m in length and 100 m to 400 m in width, and of the drainage system are purely empirical so far. The design is based on traditional experience determined by local conditions. This also applies to the heights of the brushwood fences ("Lahnungen") above the ground, which vary between 0 and 30 cm above mean tidal high water level resulting in a total height of approximately 1.00 m. The materials that have been used are timber poles with bundled willow in between (see Fig. 3). These basically permeable structures become partly impermeable by a protective earth embankment, and from the fine sediments that settle within the brushwood.

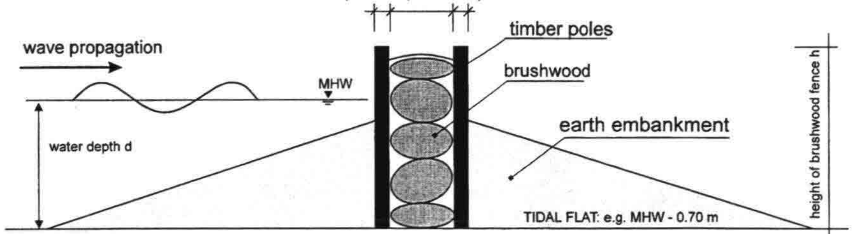


Fig. 3: Traditional Brushwood Fence ("Lahnung") with Earth Embankment

In some areas with heavy wave action and stronger tidal currents such fences were also constructed from prefabricated concrete, geotextiles filled with sand and rubblemound stones. Apart from the fact that they are impermeable, materials of this kind are today considered not in accordance with the environment, while timber and brushwood are natural materials and thus acceptable. In addition, the permeability of the brushwood fences allows the passage of water and sediment. To a certain extent, waves are damped out under mean tidal conditions (Schulz and Zimmermann, 1994).

Sediment consolidation and accelerated growth of the surface within the reclamation fields is achieved by digging small ditches about 2 m wide with a depth of approximately 40 cm. Excavated mud is placed on the areas between the ditches, thus leading to increased uploads and to further drainage of the material and of the soil underneath. The ditches, which regularly fill during rising tides, are resedimented. Repeated excavation increases the height of the fields above tidal high water levels so that the successive colonisation by glasswort, cordgrass and sea poa finally results in the salt marshes or forelands. So far, very few is known about the relationship between hydraulic parameters, boundary conditions and sedimentation. The same applies to the drainage system and its effect on consolidation. The optimum of spacing and the orientation of ditches, which should be a function of sediment, soil properties and hydraulic boundary conditions, has not so far been analysed (Schulz and Zimmermann, 1994).

3 Research Programme

In order to rationalize the discussion between engineers who feel responsible for the protection of the coast and the safety of the hinterland and the environmentalists who argue for a reduction in engineering work and maintenance, a research programme has been introduced to identify absolutely necessary protection work and minimize all man-made effects including maintenance (Schwarze and Schulz, 1995).

The research programme shall analyse and quantify

- hydraulic factors affecting mud flats and salt marshes under tides and waves
- effects of artificial works and interference on the sedimentation and erosion of forelands
- effects of the geometry of reclamation works and drainage system on waves, currents and sedimentation
- effects of reduced maintenance on foreland stability and coastal protection (aspects of grazing are not considered)

Analysis and conclusions are based on a combination of numerical simulations, hydraulic model tests, and field measurements. Field measurements were carried out in two test fields at the North-Sea coast of Schleswig-Holstein in

Germany, which were installed by the local coast administrations. Tidal currents and wave parameters were also measured by these administrations. Physical model tests in a two-dimensional flume, in a wave channel and in a three-dimensional wave basin as well as a numerical simulation have been carried out by the Franzius-Institute of the University of Hannover. The field measurements and physical model tests were the basis for the numerical simulations.

4 RESULTS

4.1 Field Measurements

Fig. 4 shows some wave characteristics, i.e. for the measured transmission coefficient at a brushwood fence in relation to the water depth. The transmission coefficient increases linear for water depths up to approximately 0.45 m above mean tidal high water level, which corresponds with h/d = 0.60 (h = height of the "Lahnung"; d = water depth) and above this remains more or less constant: The wave damping effect of a brushwood fence is of less importance for water levels above MHW.

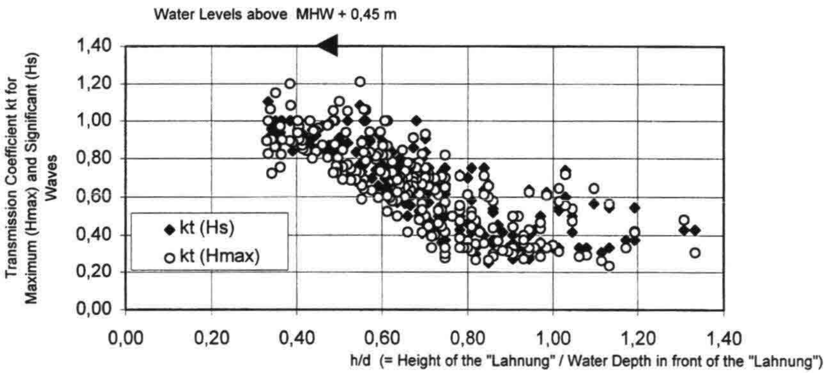


Fig. 4: Transmission Coefficient of a Brushwood Fence

for varying Water Depth from Field Measurements

4.2 Physical Model Tests in a Wave Flume and in a Wave Basin

To analyse the effects of brushwood fences on waves under varying water levels one-dimensional model tests of different brushwood fences with varying heights and shapes were performed in a wave flume (110 m length, 2.20 m width and 2.00 m depth).

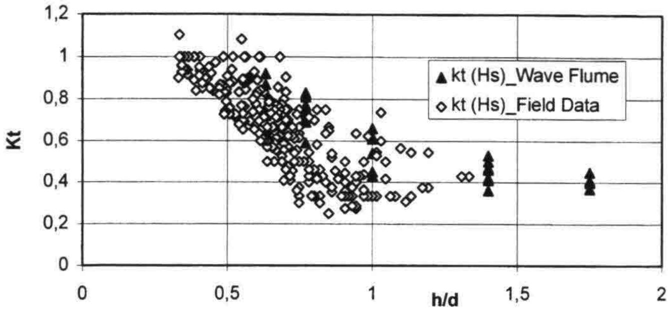


Fig. 5: Results for Irregular Wave Transmission for a Typical Brushwood Fence

(Height = 0.70 m) with a Porosity of about 20%

without a Side Earth Embankment, Results compared with Field Data

Wave transmission coefficients of these fences were obtained with rectangular regular and irregular waves. The relationship between the wave transmission coefficient and the water depth in front of the brushwood fence – as determined by field measurements (see Fig. 4) – was confirmed (see Fig. 5). As shown in Fig. 5 the transmission coefficient is only slightly higher than calculated from field data. This behaviour can be attributed to fine sediments and algae which settle within the natural brushwood fences and reduce their permeability.

To analyse and verify three-dimensional wave induced currents, hydraulic model tests with waves were carried out in a three-dimensional wave basin. The maximum water depth was 0.60 m. Waves up to approximately 0.25 m were generated. All tests were carried out with a brushwood fence with prototype scale as for the investigations in the wave flume (see Fig. 6). During the tests, video recordings of surface floats were made and current velocities were calculated.

The results compared with the numerical simulations showed satisfactory agreement for diffraction and attenuation due to the "Lahnung". Thus the numerically modelled brushwood fence is a good approximation to the prototype (see Fig. 7).

Physical model tests and numerical simulation show that the sediment-laden waters enter the reclamation fields also through the brushwood fences while thus leave the fields through the tidal opening. This behaviour has been observed also at permeable rubble mound breakwaters (see e.g. Pilarczyk and Zeidler, 1996).

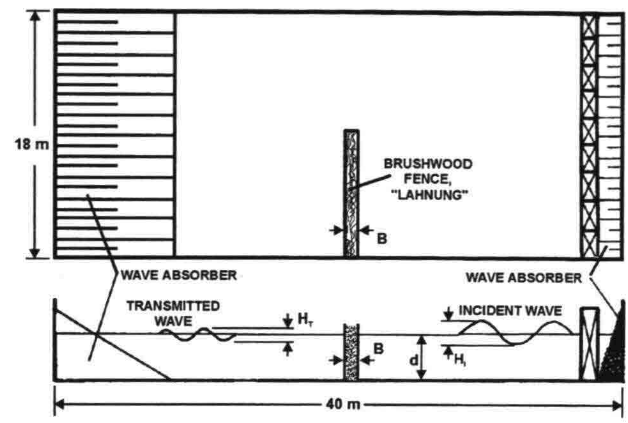


Fig. 6: Wave Basin with Brushwood Fence

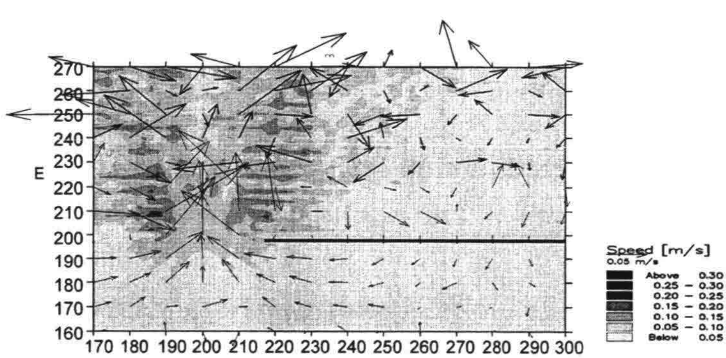


Fig. 7: Wave induced Currents computed at the Tidal Opening

and across a Typical Brushwood Fence

4.3 Numerical Simulations

To analyse longterm effects of waves and currents on sedimentation and erosion, two-dimensional numerical simulations were carried out for varying boundary conditions, i.e. waves, tides, sediment concentrations. Boundary conditions were obtained from the field measurements and from the physical models. For the simulation the **H**ydro**D**ynamic module, a wave module (**E**liptic

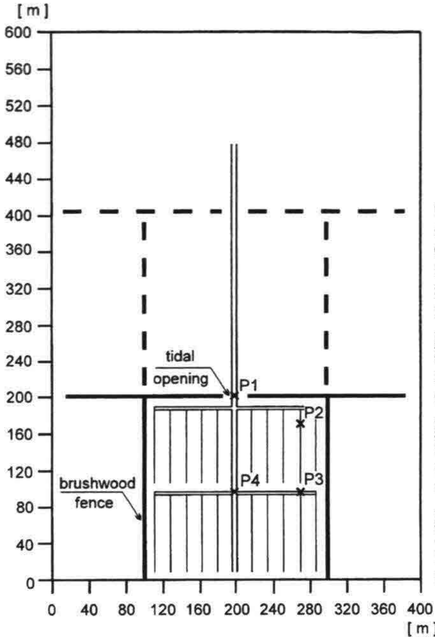
Mild **S**lope), and the **M**ud **T**ransport module of the software system MIKE 21 of the Danish Hydraulic Institute is applied. The parameter study on the influence of currents, induced by tide and waves, on sediment transport and thus sedimentation and erosion processes, showed effects of the system geometry, dimensions of the drainage system, brushwood fences and permeability of the system (see Fig. 8). The reliability of the obtained results has been confirmed via comparison with field data.

Fig. 9 shows current velocities at 4 different points distributed over the field (see Fig. 8) considering the layout of the brushwood fences as semipermeable or impermeable fences.

Fig. 10 shows the effects on sedimentation at points P1 to P4 of a sedimentation field for the different fences. In all test cases sedimentation starts immediately at the beginning of the tidal period. The highest sedimentation rates occur during the high tide period (Matheja et al., 1997).

The width of the tidal opening (results for tidal openings of 25 m and 90 m are shown in Fig. 11) seems not to be significant for the sedimentation in the fields. Only a small increase in the sedimentation rate is indicated for a tidal opening of 90 m.

For brushwood fences with earth embankments, sedimentation decreases at all points. Earth embankments combined with a regular drainage system lead to an increased sedimentaton at point P2, while this combination does not show significant effects for a sedimentation in the field centre around point P4. The conclusion is, that ditches dissipate the sediment-laden waters into sheltered areas of the sedimentation fields (e.g. at point P2), but they do not have significant influence on the sedimentation in the centre of the fields.



Boundary Conditions
for Results presented in Fig. 9 to 12:

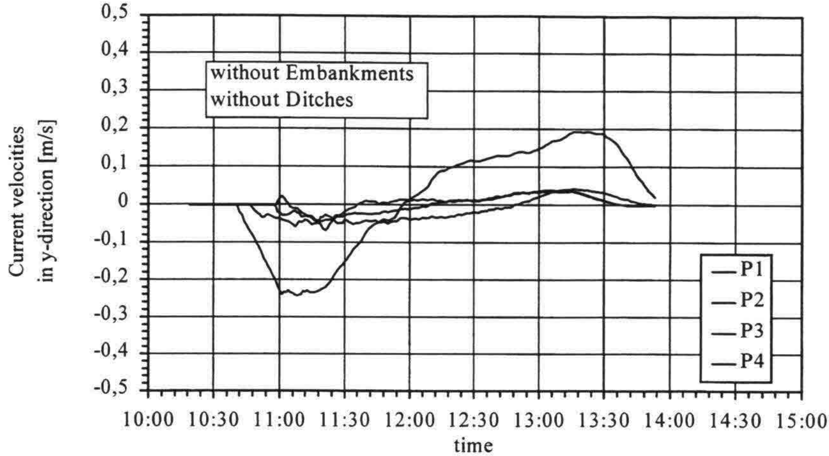
tidal opening: 25 to 90 m
ditches: main ditch; with or without small ditches
embankment: with or without earth embankment
number of fields: one or two
slope of bottom: 1:800
permeability of brushwood fence: 20%

<u>HD-Module:</u>
tidal curve: mean tidal curve of the test fields
direction of currents: perpendicular to the coast

<u>EMS-Module:</u>
wave heigths: 10 to 23 cm
wave period: 3 s
wave direction: perpendicular to the coast

<u>MT-Module:</u>
initial concentration: 350 g/m^3
mean settling velocity: $7.3 \cdot 10^{-6}$ to $7.3 \cdot 10^{-5}$ m/s
sediment fraction: referring to test fields

Fig. 8: Numerical Model Area and Boundary Conditions



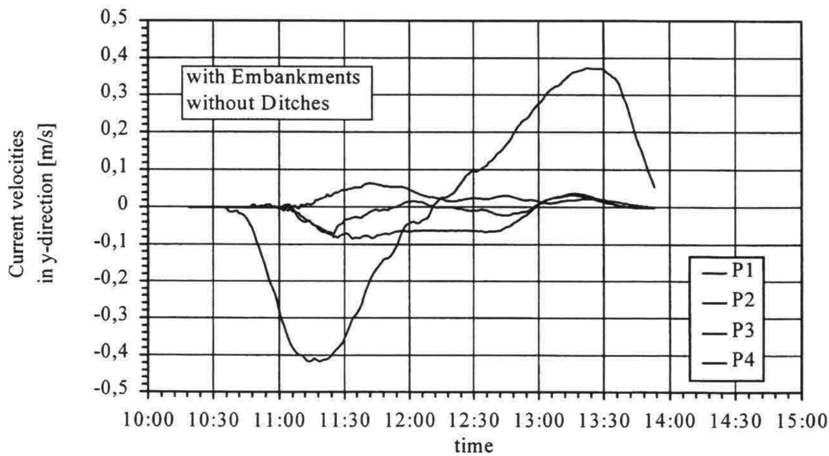
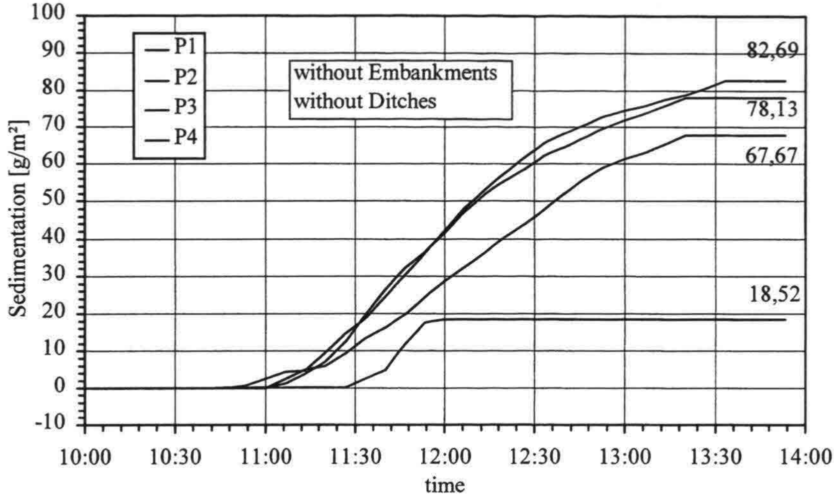


Fig. 9: Current Velocities at Reference Points in the Test Field

due to Different Layouts of the System



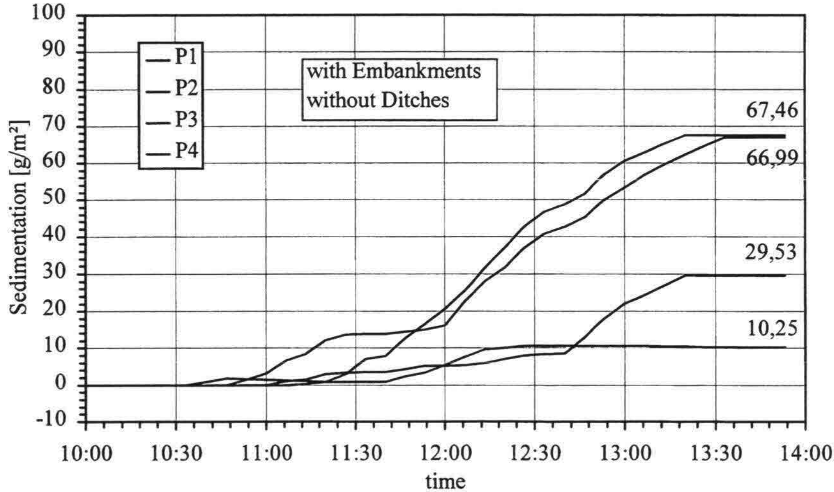
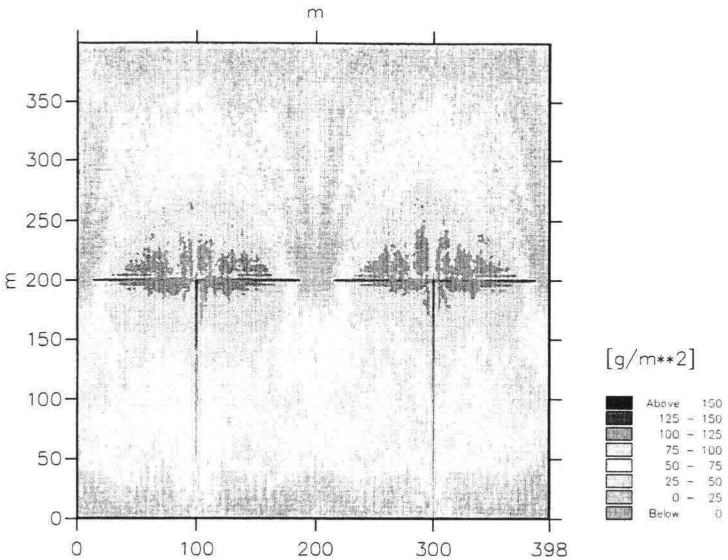


Fig. 10: Sedimentation at Reference Points in the Test Field

due to Different Layouts of the Brushwood Fences



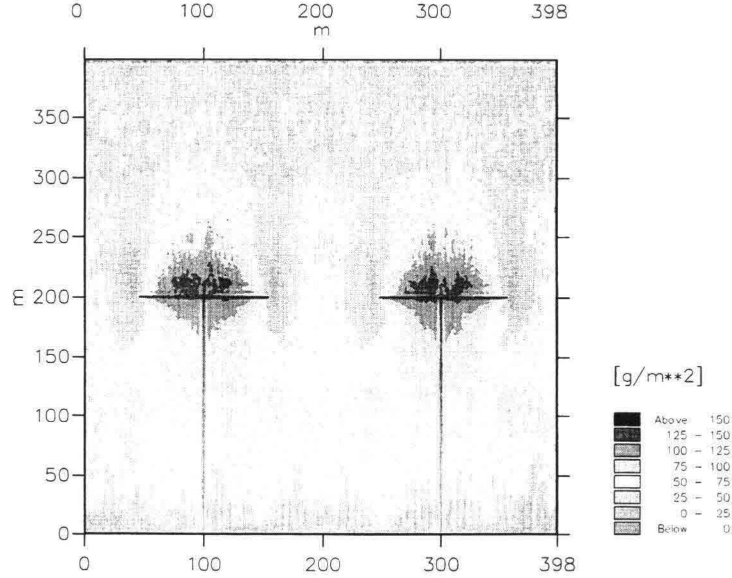


Fig. 11: Distribution of Sedimentation in the Model Area after one Tidal Cycle due to Different Opening Widths

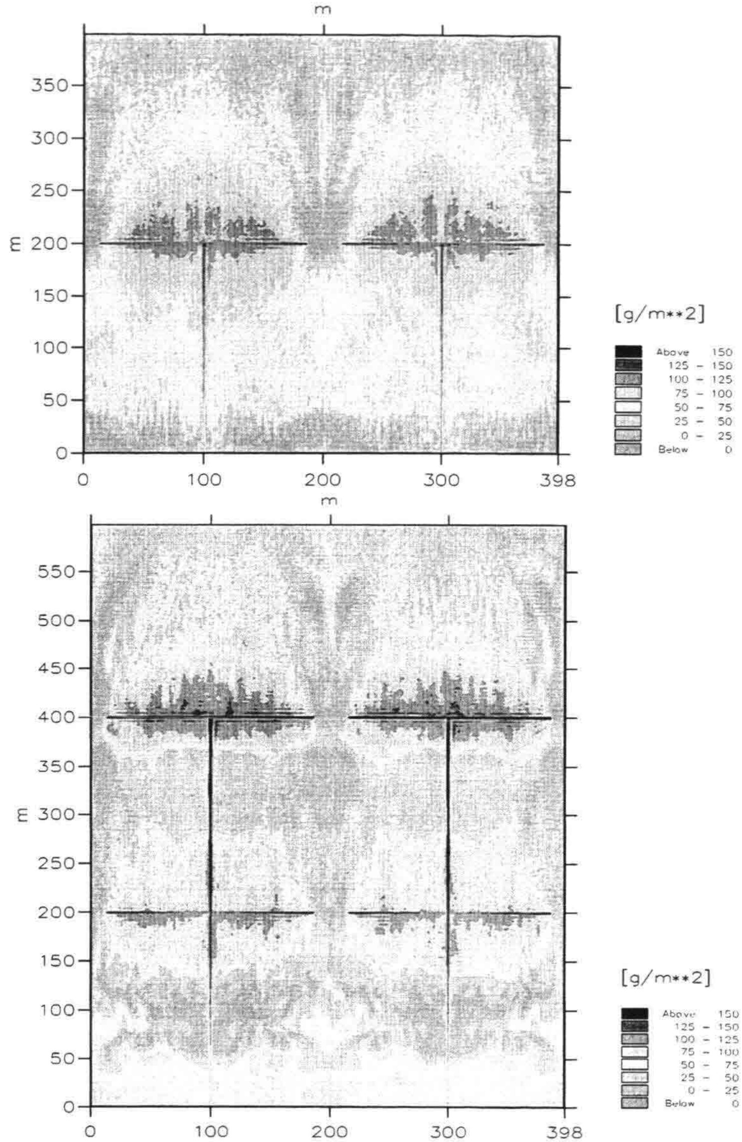


Fig. 12: Distribution of Sedimentation in the Model Area after one Tidal Cycle for a One-Field-Geometry and a Two-Field-Geometry

With respect to an optimised design of land reclamation fields for a sustainable foreland development the main results of the study (sedimentation rates) are:

– sedimentation decreases for increased **tidal openings** in the area of the tidal opening; but there is sedimentation if a drainage system with ditches is introduced

– in all cases where **ditches** are dragged into the fields sedimentation increses in comparison to the cases without ditches

– if there is no **earth embankment** less sedimentation takes place in front of the reclamation field

– for single reclamation fields with ditches there is no evident influence of an earth embankment

– for single reclamation fields with an embankment and without ditches the sedimentation rates decrease

– there are no evident differencies between the sedimentation rates of **one-field-geometries** and **two-field-geometries** with embankment

– in the area of the tidal opening sedimentation decreases if a second field is placed in front a single one – especially for larger tidal openings (see Fig. 12).

5 Summary

Increase of the level of certain areas of the Wadden Sea through enforced sedimentation in front of the main dikes by introducing sedimentation fields surrounded with brushwood fences („Lahnungen") is confirmed from physical and numerical simulations with the verification from field measurements. Effects of oblique waves and tidal currents and extreme water levels (storm tides) are subject of ongoing research.

6 Acknowledgements

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High Water Protection in Coastal Towns of Mecklenburg-Vorpommern

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Abstract

Considering extreme events storm water levels at the Baltic Sea of Mecklenburg-Vorpommern can rise up to 3 m above mean water being connected with strong wave attack. Since there is almost no tidal influence in the Baltic Sea, the duration of high water levels is directly connected with storm duration. Storm water levels can last several days.

Due to the natural topography an area of 1050 km² in the coastal regions of Mecklenburg-Vorpommern is endangered by flooding, among this an area of 75 km² in the coastal towns Wismar, Rostock, Stralsund and Greifswald.

Since in the past, high water protection works were carried out in the outer coastal areas, which are exposed to high wave energy, the coastal towns are showing a very poor protection level.

Material damages of 2.3 thousand million Deutsch Marks (1.3 thousand million US $) are expected in the four coastal towns by an extreme high water level according to an analysis. This shows the need of further protection works. For this reason the Federal State of Mecklenburg-Vorpommern is executing investigations for the high water protection measures in Wismar, Rostock, Stralsund and Greifswald.

Possibilities of realisation of different concepts based on dikes as well as on high water barriers are currently being discussed. Besides economical aspects, ecological conditions and town planning limitations must be considered. Various intersections between high water protection and planning of cities and their infrastructural development are existent.

Two different levels of administration are being affected, where aspects of high water protection are in the responsibility of the Federal State of Mecklenburg-Vorpommern and all other planning works have to be executed by the towns. This results in a high degree of necessary communication between both which can lead to complications according to experience.

1 High water events in Mecklenburg-Vorpommern

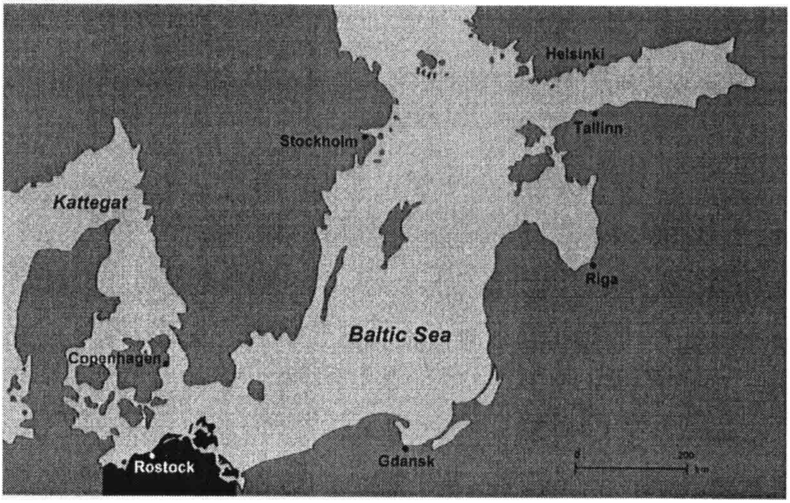

Fig. 1: The Baltic Sea

Despite the fact that the Baltic Sea is only a small inlet of the Atlantic Ocean, to which it is connected by three shallow straits, dangerous storm water levels can still occur there. The long-stretching basin of over approximately 1,200 kilometres to the northeast provides favourable precondition for them (Fig. 1). In the case of storms from northeasterly directions, high water on the coast of Mecklenburg-Vorpommern can surpass even two metres. After long-lasting westerly winds, which lead to the induction of water from the North Sea, the Baltic Sea can reach a capacity of up to 0.5 metre above the mean sea level. In addition, suddenly changing winds, especially from a southwesterly to a northeasterly direction, cause oscillation in the Baltic Sea basin, which can also lead to increased water levels.

When such problems occur simultaneously, then storm water levels of more than 3 metres above the mean sea level are possible. Table 1 shows the worst high water events of the past 125 years. The stated water levels for the coast of Mecklenburg-Vorpommern were established on the basis of the peak levels of the high water event in 1872. An additional sum has been placed on the peak levels of 1872 to account for the centennial changes in the sea level (15 to 25 centimetres every century). This is essentially a relative increase in the sea level, resulting from the fact that the southerly Baltic coast is sinking, while the Scandinavian coast has continued to rise since the Ice Age.

Date	Wismar m MSL	Rostock -Warnemünde m MSL	Greifswald m MSL
Design-Water-Level	3.20	2.85	3.00
13.11.1872	2.80	2.43	2.64
31.12.1904	2.28	1.88	2.39
04.01.1954	2.10	1.70	1.82
30.12.1913	2.08	1.89	2.10
04.11.1995	1.98	1.58	1.77
07.11.1921	1.96	1.50	*
02.03.1949	1.74	1.50	1.80
12.01.1987	1.69	1.40	1.41
25.11.1890	1.67	1.48	*
11.12.1949	1.64	1.29	0.84
15.02.1979	1.57	1.27	0.98
09.01.1914	1.57	1.60	*
14.12.1957	1.56	1.35	1.52
12.01.1968	1.55	1.50	1.54
14.1.1960	1.55	1.18	1.13
21.02.1993	1.52	1.29	1.45
19.04.1903	1.52	1.25	1.29

Table 1: High water events at the coast of Mecklenburg - Vorpommern with water levels of more than 1.50 m above mean water level (MSL) since the year 1872 (* no information)

Table 2 shows the design water levels at the coast of Mecklenburg-Vorpommern. It is noticeable that the design-water-levels for the bays (Bodden) and lagoons are considerably below those for the prevailing coastline. The causes are the low induction averages in these waters, which are however only evident when the coastal protection installations on the prevailing coastline are fully operational. This means that there must not be a breach of the coastal line.

Place	Coastline	Design-Water-Level m MSL
Wismar	Wismarbucht	3.20
Rerik	Baltic Sea	3.00
Warnemünde	Baltic Sea	2.85
Rostock	Warnow	3.00
Born	Saaler Bodden	1.65
Zingst	Baltic Sea	2.70
Zingst East	Barther Bodden	2.05
Arkona	Baltic Sea	2.30
Saßnitz	Baltic Sea	2.40
Mukran	Baltic Sea	2.40
Stralsund	Strelasund	2.70
Greifswald-Wieck	Greifswalder Bodden	2.90
Ahlbeck	Baltic Sea	3.00
Koserow	Achterwasser	1.85
Ueckermünde	Kleines Haff	1.75

Table 2: Design-water-levels at the coast of Mecklenburg-Vorpommern

Year	≥ 1,00 m MSL	≥ 1,00 m MSL
1872	40 h	19 h
1904	27 h	22 h
1913	62 h	35 h
1949	8 h	7 h
1954	14 h	5 h

Table 3: Period of dwell for two water level ranges of some high water events on the depth gauge in Greifswald

Since the Baltic Sea is entirely tideless, the duration of the high water events is directly determined by both the duration of the storm occurrences which cause them and their wind directions. As a result, high levels of water can prevail for several days. Table 3 shows the period of dwell for two water level ranges of some high water events on the depth gauge in Greifswald.

Fig. 2 shows the depth gauge and wind measurements recorded by the stations in Greifswald during the occurrences of 1872. It can clearly be seen, how as a result of southeasterly storms on 7th November, there was a drop in the mean sea level of over 0,5 metre, before it had conversely risen again by 11th November to 0,5 metres above the mean sea level, due to now decreasing winds. The occurrence of northeasterly storms thereafter produced the steep course of peaks.

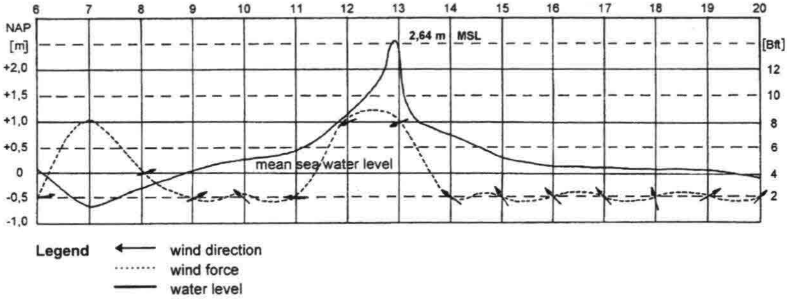

Fig. 2: Depth gauge and wind measurements recorded by the station Greifswald between the 6th and the 20th November 1872

2 Damages due to high water events in Mecklenburg-Vorpommern

Since storm water events are only dangerous if they can flood low-lying coastal areas, an analysis of the height proportions of the coastal region is necessary in order to assess the dangers of high water levels to our coast. According to natural topography, about 105,000 hectares are potentially areas in danger of being flooded, 7,500 hectares of which are coastal towns. Without coastal protection provisions, it is estimated that severe flood tides would cause damage in these areas of 3.2 thousand million Deutsch Marks (1.8 thousand million US $). Table 4 shows that the biggest potential damages are apportioned to the four big coastal towns (2.4 thousand million Deutsch Marks; 1.4 thousand million US $), because these contain the largest concentration of wealth in the flood areas. Fig. 3 shows these areas for Greifswald as an example.

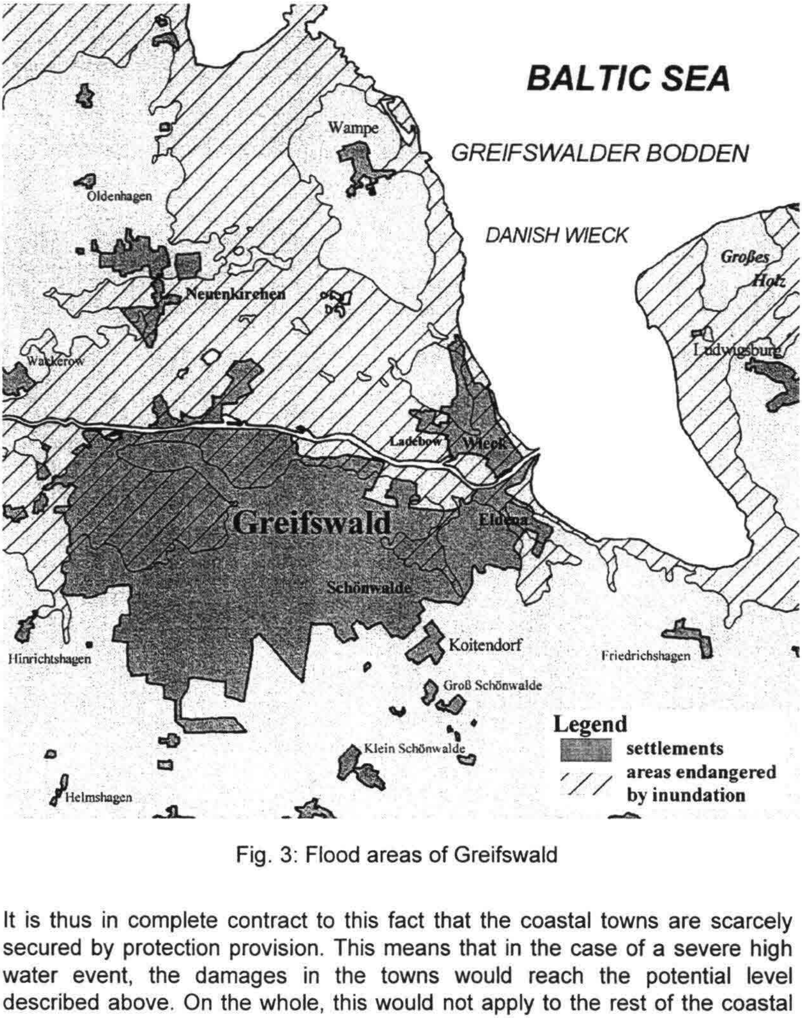

Fig. 3: Flood areas of Greifswald

It is thus in complete contract to this fact that the coastal towns are scarcely secured by protection provision. This means that in the case of a severe high water event, the damages in the towns would reach the potential level described above. On the whole, this would not apply to the rest of the coastal regions, as there are numerous coastal protection provisions elsewhere.

Towns / Districts	Areas endangered by inundation in km^2	Endangered people	Endangered workplaces	Expected material damages Mio DM (Mio US $)
Towns				
Wismar	71,700	7,653	6,888	260 (147)
Rostock	455,100	64,036	29,353	1,472 (832)
Stralsund	19,400	9,843	4,320	200 (133)
Greifswald	203,000	17,461	12,122	505 (285)
Districts				
Nordwest-Mecklenburg	341,900	2,893	646	46 (26)
Bad Doberan	654,700	8,782	2,402	113 (64)
Nord-vorpommern	2,842,000	19,159	5,376	244 (138)
Rügen	1,887,100	9,075	1,680	121 (68)
Ost-vorpommern	3,431,600	18,890	5,724	258 (146)
Uecker-Randow	578,600	5,101	1,914	56 (32)
Coastal Area of M/V in total	104,851,000	162,893	70,425	3,275 (1850)

Table 4: Estimated damages due to high water events in Mecklenburg-Vorpommern without any protection measures

As a result, there is an increasing need to act in this area. This should not, however, detract from the future necessity of securing the prevailing coast as the first priority. After all, severe swell has an effect on the coast when storm surges occur, just like the water level. Without continual work on coastal protection, the Baltic Sea would soon penetrate the bay (Bodden) waters, thereby leading to the formation of islands (Fig. 4).

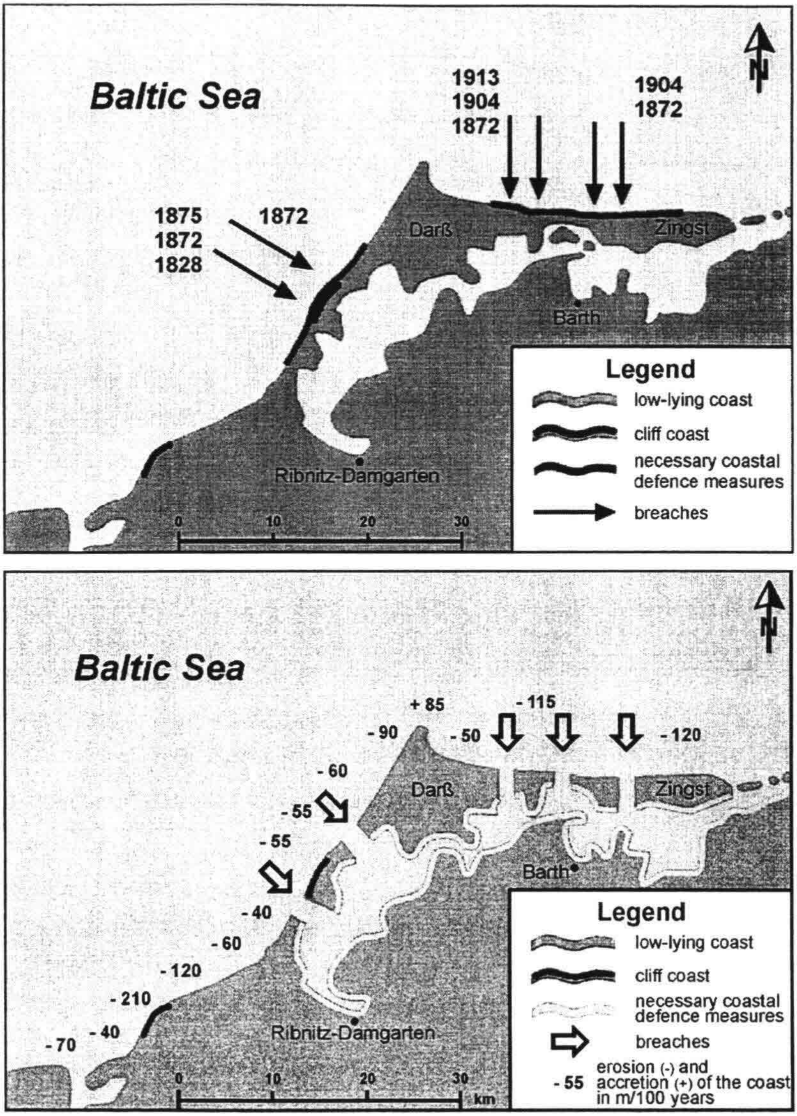

Fig. 4: Future situation without continual work on coastal protection: Penetration of the Baltic Sea into the bay (Bodden) waters and the formation of islands

Defending the settlements on the coast when penetrating water motion occurs is scarcely possible. As well as material assets, human lives are also directly at risk. Due to this danger which has existed for a long time, coastal protection work on the prevailing coast has historically increased. It has become a long-term project owing to the permanent regressive tendencies of the majority of our coast. In particular, continual reinforcement of the dunes is necessary, since they are the predominant coastal protection devices in flat coastal areas.

In the towns, however, which have developed considerably during the last decades, there has been a clear increase in material assets and with it an increase in the need for protection. Since severe swell is scarcely a problem here and the population can easily flee to safety, human lives are not normally endangered. Nevertheless, the protection of the coastal towns against storm water levels is not only an economic necessity, but it is also laid down in the Regional Water Law, which actually binds the region to make provisions for coastal protection from high water.

3 High water protection measures in coastal towns of Mecklenburg-Vorpommern

As a result, appropriate measures for the towns of Wismar, Rostock and Greifswald have been in preparation since 1994. The major problem in the planning of high water protection measures for urban areas is the lack of space. Built-up areas often extend right up to the waterfront. Building dikes is therefore only a limited possibility. In many cases a alternative constructive solutions in the form of walls and sheet piling will have to be considered. Furthermore, coastal towns are characterised by the fact that they have a harbour. As a rule, the road and rail approach lines therefore cross the line of the high water protective device. This requires special locking mechanisms (sliding gates, lifting gates, dam-beam seats, etc.) Consequently, there are always positions in the line of defence, which are at considerable risk and therefore require special handling when they close. The number of such interconnections should therefore be kept fundamentally low, or at least overlaps foreseen.

In conceiving the protection installations, the provision of several independent systems is foreseen, whereby each system protects an individual part of the town. This leads to greater security of the entire protection and enables its realisation in short phases of construction. Yet this action is mostly not compatible with urban sewage systems, because they represent subterranean connections between the individual protection systems. This can only be prevented by extensive precaution measures in the sewage installations.

It is particularly problematic that the regional authorities plan and carry out the high water protection measures, whilst other urban planning is a matter for the municipal authorities. As a rule, this leads to conflict with town-planning, because in the phase of conceptional pre-examinations, there is no possibility to foresightedly secure the lines needed for high water protection in the inner city area, as exactly those areas of land lying close to the water are especially attractive and are foreseen for diverse usages. The preservation of the importance of coastal protection, which is carried out in the interests of the towns, therefore necessitates intensive contact and co-ordination between the authorities concerned. Since the towns are now involved in a phase of intensive development, with numerous projects which interact with high water protection being planned or carried out the basic conditions for the planning of high water protection are therefore constantly changing. This will lead to serious problems once the planning reaches a more concrete stage, (project implementation as opposed to mere concepts). Due to the limited capacities of the regional authorities responsible for coastal protection, the planning and realisation of high water protection measures will stretch over a lengthy period of time. Since the towns are hardly prepared to postpone their plans for so long, the coastal protection plans have to be adapted to the constantly changing conditions. This hinders the realisation of the proposals quite considerably.

3.1 Hanseatic Town of Greifswald - an example of high water protection measures in Mecklenburg-Vorpommern

While the planning for the towns of Wismar and Rostock is still at the stage of pre-examination, the work for Greifswald is already well-advanced. All further comments are therefore limited to this town.

The results of an enquiry which was carried out at the beginning of the examinations researching potential damages clearly show the need to act. If there was a high water event with a water level of 3.0 metres above the mean sea level, then of the areas flooded, 57% would be under 1.0 metre of water, 21% would be under 1.0 - 1.5 metres and 22% under more than 1.5 metres of water. The damage incurred would be about 505 million Deutsch Marks (285 million US $).

In contrast to the other towns, Greifswald has been thinking about creating an effective high water protection device for quite some time. Efforts were already being made in the 1950´s to protect Greifswald with the building of a high water barrier in the Ryck. The project did not stop at the planning stage and in 1956, work began on the adjoining dike on the west bank of the Ryck in front of Wieck. Although the high water barrier itself was never built due to financial reasons, the idea remained in the minds of hydraulic engineers.

Thus, when in 1992, work began on a general plan for coastal protection in Mecklenburg-Vorpommern the high water barrier was included on the list of emergency measures to be considered.

At the beginning of 1994, planning work for the high water protection of Greifswald began. Even though the option of a high water barrier was a favourable one due to the natural conditions, it was not the only solution considered. The possibility of building a system of dikes along the Ryck was examined too. As sections of dikes had already been built along the Ryck this solution presented itself too. These dikes, however, are not big enough to enable their extension. As a result, a strengthening of those dikes already there, as well as the construction of new dikes so as to create a continuous line on both banks of the Ryck, had to be examined.

The construction of a high water barrier entails a complete change in the scenery at the spot were the locking mechanism is situated. In this case, it spoils the especially beautiful view of Wieck from the bay (Bodden). In order to minimise this imposition, work at the beginning of the examinations focused on a stop gear lock, which completely sealed off the Ryck without restricting the sides and which sinks outside the blocking times and is then no longer visible. Two sector locks, each 30 metres wide, came into question as the technical solution. Yet it quickly became evident that such a device clearly exceeded the existing possibilities, both in its construction and its operation and maintenance. The options of a 30 metre miter gate with sliding side doors and a 30 metre sector lock were revised to create an option which was favoured henceforth, namely lateral dams and 18 metre miter gates. Safety and technical criteria in particular were the decisive factors. Absolute priority must be given to them above all else. This means, for example, that a double pair of miter gates is foreseen and a special locking device has to be designed, because if there were a high water event, ice could prevent the miter gates from closing. They must therefore close once the ice starts to form. In the bay of Greifswald (Greifswalder Bodden) this happens almost annually.

Henceforth, a balance must be found between these technical necessities as well as cultural needs and other public interests such as shipping. This mean among other things, the most inconspicuous integration of the high water barrier into the countryside, the guarantee of the biological perviousness of the Ryck when it is closed due to the formation of ice, the enablement of the fields bordering the Ryck to flood up to a certain high water level and the ecologically harmless reinforcement of peripheral connecting dikes.

As well as the fact that to build the high water barrier, no intervention in the inner city area is needed, a further advantage of this option is that it guarantees the high water safety of the entire low country around the Ryck. It thereby creates more favourable conditions for the further development of the town of Greifswald than is otherwise possible with the dike solutions.

With regard to the high water barrier, different problems materialise when considering solving the problem with dikes. These mainly result from the need for space, which is required when building a dike and from the impairment caused to the town's image when a continuous line of construction with a maximum height of about 3.5 metres above sea level, is erected near the river banks.

As a result of the lack of space, it was necessary to design individual sections of the line as a quay. A completely satisfactory solution for the safety of the borough of Wieck could not be found, since raising the quay installations to a height of 3.5 metres here would destroy the fundamental characteristics of this part of the town. Securing the industrial estate around Eldena Harbour is also not possible for the same reason.

Despite these disadvantages, this option does at least offer a high degree of security, since the whole system consists of several independent components. The dikes are also relatively immune to ice and they can also be easily defended if there were a storm water level. Furthermore, they need little maintenance and their additional reinforcement is not a major perspective problem, unlike the high water barrier which makes certain precautions necessary, even when being built.

From a budgetary point of view, the building of the dikes could be financed over a period of many years, while a high water barrier represent a major investment, which would account for a large proportion of all funds available for coastal protection.

This short comparison already makes it clear that the are pros and cons to both options. Clearly favouring just one option is not possible, especially as according to initial estimates, both would cost about the same. Giving one option preference over the other depends entirely on the importance of the individual assessment criteria. In considering this, the opinion of the town in question should play a role, particularly as this is itself of great importance in light of the authorisation procedure to be formally carried out with public involvement before one of the option is chosen. As a result, the documents compiled so far has been passed on the Hanseatic Town of Greifswald for examination and analysis. Only when this analysis has been presented will a suggested preference be made known before being forwarded to the Minister for Construction, Regional Development and Environment of the Federal State of Mecklenburg-Vorpommern, when a decision will be made.

Such action is, however, not at all probable in the other towns, since after initial examinations, high water barrier solutions are scarcely feasible. The possible options will consequently be limited to various lines and technical solutions when designing linear-based protective installations.

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Topic II

Field Investigations

Measurements of Waves

Chairman: Hwung-Hweng Hwung

The effect of the irregularity of waves for the design of coastal structures

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Abstract

Sea waves in nature are irregular in time and space, whereas the development of theories and basic research usually requires the assumption of regular long crested waves.

For design the knowledge from regular waves has to be implemented by using simulation methods in time and frequency domain (e.g. linear superposition method, single wave statistics). As such methods are again based on simplified solutions, their general validity and reliability has to be proved by measurements in nature, physical model tests or numerical/theoretical approaches.

In the paper various simulation methods are summarised and examples are worked out, to demonstrate the degree of importance for various problems.

1 Introduction

Sea waves, especially wind waves, in nature are rather irregular, as well the surface elevation as it is to be seen or measured, as the related velocity or pressure field below the surface, which is the intrinsic reason for the action on shorelines and structures in the sea.

Analysis and design methods are based on information from measurements of the surface elevation. The connection to the physical processes has to be established from theoretical investigations or measurements in hydraulic models or nature.

The term irregular waves is used here in contrast to regular long-crested waves, which also can be termed 'waves of permanent form'. Most of our perception and knowledge on wave related processes is based on experiments with regular waves and regular wave theories, and inevitable analysis, simulation and design methods are related to regular wave knowledge.

The straightforward design concept of representing the irregular sea state simplified by a significant regular long-crested wave has a long tradition. However, the wave parameter selection only on the basis of surface statistics (instead on the basis of the statistics of the relevant physical process) and neglecting directional features, may result in misjudgement. Design methods and simulation methods considering effects of the irregularity of waves are discussed in the following.

2 Description and simulation of sea waves

A useful model to deal with sea waves and to schematise methods for analysis and simulation or design is the superposition model. The sea state is seen as superposition of regular long-crested wave components of various frequencies (periods), amplitudes (heights) and directions. If only sinusoidal wave forms are considered and non-linear interactions are neglected it is a linear superposition model.

In the frequency domain the general case of the short-crested irregular sea is represented by a directional spectrum with energy density distribution in frequency and direction (Fig. 2.1).

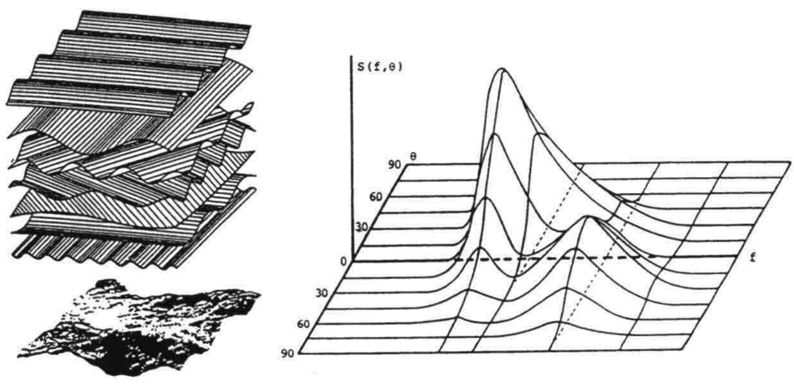


Fig. 2.1: Superposition model and directional spectrum (SCHADE, 1991)

The measurement of the complete directional spectrum in nature or physical models requires information on surface elevation and directional components (e.g. floating buoys with measurements of slope or horizontal acceleration, combinations of pressure cells with velocity meters, gauge arrays). By analysis methods in frequency domain a certain (but not complete) information on the frequency dependent directional energy distribution can be estimated. Analysis of directional parameters in time domain is not impossible, but not usual up to now.

If only the time dependent water-level fluctuation (time-series) at one point is measured (e.g. floating buoy with measurement of only vertical acceleration, pressure cells), directional properties cannot be distinguished. A frequency domain analysis of the time-series results in a frequency spectrum, usually given as spectral density. The analysis of the time-series in time domain (usually with zero-crossing definition for the single waves in the irregular wave train) results in a number of single wave events (wave heights and related periods).

In terms of the superposition model this corresponds to a first step of simplification where all components are assumed to have the same (average) direction. In space this is a long-crested irregular sea (Fig. 2.2).

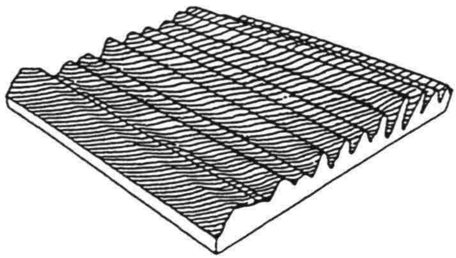


Fig. 2.2: Long-crested irregular wave train: the first step of simplification (EAK 1993)

The second step of simplification is the averaging of all frequencies (periods) and amplitudes (wave heights) and representation of the sea state by one single regular wave of permanent form.

From the steps of simplification of the superposition model it becomes evident that effects of the irregularity of waves are due to directional irregularity, irregularities in the sequence of wave heights and periods and the profile of the single wave (shape irregularity).

The analysis of measurements can be performed in time and frequency domain. As simulation and design methods are strongly related to the analysis, various methods in time and frequency domain features and parameters are used in basic and applied research.

Analysis methods determine the features of the sea state and related parameters from measurements. Simulation methods use the features of the sea state and/or related parameters determined by the analysis methods and end up with design methods or diagrams, which give the relations between surface parameters and physical quantities (e.g. forces, transport quantities, modification of wave features).

The analysis in frequency domain is based on the Fourier transformation of measured quantities. The wave height parameter H_{m0} results from the total energy of the frequency spectrum. Typical wave period parameters are T_p (peak period) or $T_{0,1}$ and $T_{0,2}$ as „average periods". Parameters to characterise the shape of the spectrums are not so widely used, and mostly for design a theoretical energy density distribution (e.g. JONSWAP, TMA) is assumed. Common directional parameters are the mean direction as a function of frequency $\Theta_m(f)$ and parameters (s or Ψ) for the directional spreading function $D(f,\Theta)$. Again for design directional spreading functions (e.g. SWOP, MITSUYASU-type) are available.

The analysis in time domain is based on the distribution of individual wave heights and periods (up to now only to be applied on the time-series of the surface elevation without directional attributes). The complete distributions can be given in histograms. The RAYLEIGH distribution is the most often used theoretical distribution for the wave heights, as long as no wave breaking has occurred. Wave height parameters are e.g. H_{max}, $H_{1/10}$, $H_{1/3}$ and H_m. The period parameters are either related to the wave height parameters ($T_{H1/3}$ etc.) or directly calculated from the period distribution ($T_{1/3}$ etc.).

The simulation methods are closely related to the analysis methods. The following methods are generally used:

- theoretical simulation in frequency domain (spectrum and transfer function)
- theoretical simulation in time domain (probability of individual waves)
- measurements in irregular waves (nature, hydraulic models, numerical models).

The representation of the sea state by a (significant) regular wave is not seen as simulation method, but more a design method. For design problems where the maximum wave establishes the design condition, this procedure is obvious, although the real wave may differ in profile and impact from a wave of permanent form. Using a regular wave with wave parameters of the significant wave (e.g. $H_{1/3}$, $T_{H1/3}$) in investigations (hydraulic or numeric models, theoretical calculations) may possibly contain an error, depending on the type of wave problem. This is especially to be considered in case of a strong directional dependence (diffraction, reflection) or not linear coherence between wave heights and the physical process under investigation (e.g. wave overtopping, reduction of wave energy due to breaking). Typically, when results are energy dependent (e.g. transport quantities of sediment) H_{rms} (root mean square wave height) is a better parameter. Using significant parameters resulting from proper simulation methods, however, is a correct procedure.

The simulation method in frequency domain is based directly on the conception of the linear superposition model. The design wave spectrum (either directional spectrum or frequency spectrum) is divided into single wave components. Each component is treated as independent wave train. The alteration of magnitude and phase by a physical process is determined from linear wave theory or otherwise existing knowledge on linear waves. The result spectrum is the amount of the altered single components.

The method is generally termed transfer function method and can be applied to the general directional sea and the long-crested irregular waves (first step of simplification). Basic requirement is the sufficient linearity of the treated process.

Further developments of this method include non-linear interactions between the single wave components.

The usual application of the simulation method in time domain is restricted to the simulation of long-crested waves, although in principle a time domain representation of the general case of the directional sea state is not excluded. Each wave of a given design wave train is treated as independent regular wave with parameters H and T. Sometimes the use of other surface quantities (e.g. half-wave parameters) improves the accuracy of individual results. The alteration of the wave height or the physical process under investigation can be determined from wave theories or otherwise existing knowledge, and allows the consideration of non-linear effects and strong interactions (wave breaking). The simplification is the assumption, that each wave is treated independent and has the shape of a steady wave. Practical calculation is mostly performed not on single wave events, but on classes of wave heights with related periods and probabilities.

Further developments of this method try to consider the influence of preceding waves.

Hydraulic model tests are used as simulation method when no sufficient knowledge is available as basis for the usual simulation methods, when the physical processes are obvious too complicated, or simply for verification or development of simulation methods. Investigations with long-crested irregular waves in channels or basins are standard. Meanwhile also a number of facilities is equipped with directional wave generators. The wave generation in hydraulic models requires the use of simulation methods in frequency domain. Especially when steep waves, waves with special features (e.g. defined wave grouping), or measurements from nature are to be reproduced, non-linear wave generation theory or iterative methods have to be used. The generation of directional sea state is more complicated and often only a limited area of unique wave characteristics may be available.

3 Selected examples and remarks

3.1 Shoaling and refraction

Shoaling is a physical process without influence of directionality. As long as waves are in a relative water depth of $h/L_0 < 0.1$, the variation in the shoaling coefficient is small. Each of the simulation methods in frequency and time domain and also the representation by a significant wave height and period results in sufficient accuracy for practical purpose. In very shallow water and close to wave breaking the simulation method in time domain can be used to consider non-linear shoaling.

Refraction is a physical process with only moderate non-linear effects as long as slopes are not to steep and no extreme concentration of energy is forced due to strong bottom curvature. The influence of directionality is not so strong as in diffraction, but has a certain influence.

For a coast with straight and parallel depth contours GODA, 1985 has worked out diagrams based on the superposition method, from which the directional influence on the refraction coefficient K_r (Fig. 3.1) and the predominant direction can be seen. The difference between regular wave refraction and refraction of a directional spectrum with s_{max} = 75 (Swell with long decay distance and relatively small wave steepness) is less than 3%, so that the results for s_{max} = 75 may be taken approximately as regular wave results to judge on the influence of directional spreading. As the predominant direction is even minor influenced, regular wave results with significant parameters are a reasonable approximation.

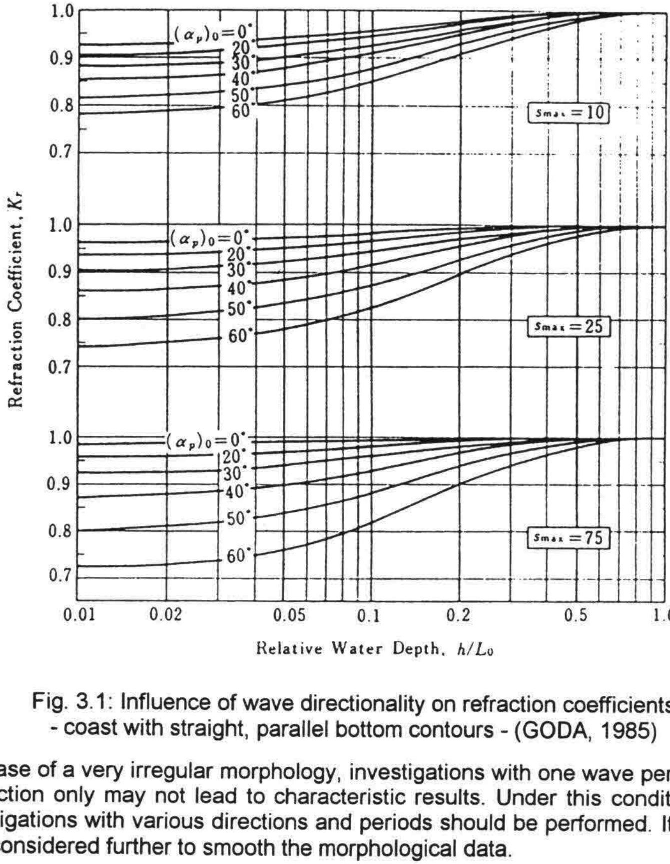


Fig. 3.1: Influence of wave directionality on refraction coefficients - coast with straight, parallel bottom contours - (GODA, 1985)

In case of a very irregular morphology, investigations with one wave period and direction only may not lead to characteristic results. Under this conditions investigations with various directions and periods should be performed. It should be considered further to smooth the morphological data.

In case of a very oblique mean wave direction, additionally the directional spreading of the local input spectrum has to be discussed. The energy in the landward part may be reduced, compared to an open sea directional spectrum.

3.2 Shallow water wave breaking

Wave breaking is a strongly non-linear interaction and therefore the simulation method in the time domain is generally used. Regular wave information has to be taken from hydraulic model tests. Details of the simulation methods and the evaluation of the results have to be performed by hydraulic model tests or measurements in nature.

A number of simulation methods exist (e.g. GODA, 1985, BATTJES/JANSEN, 1978, THORNTON/GUZA, 1983, STROTMANN et al., 1991, GÖTSCHENBERG/DAEMRICH, 1987) with various assumptions on shallow water breaking criteria for regular waves and on the change of the histogram of the incoming waves. As an example a diagram from GODA, 1985 is given in Fig. 3.2. It is easy to be seen, that the representation of the breaking process by one regular wave criterion as e.g. H/h = 0.78 does not represent the process in the whole area. The representation by a fixed relationship $H_{1/3}/h$ requires already the knowledge on results from measurements or simulation methods.

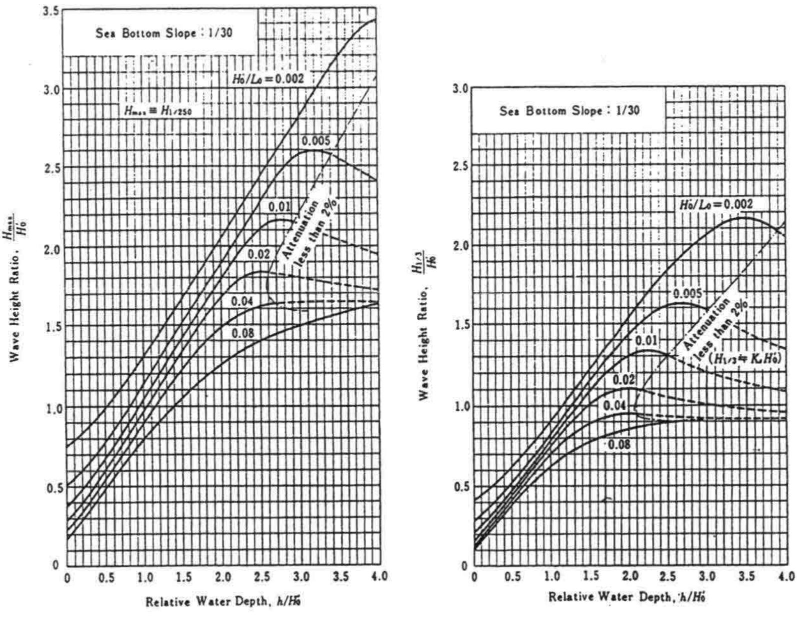


Fig. 3.2: Wave height in the surf zone (GODA, 1985)
Bottom slope 1/30

In numerical parametrical models based on the simulation method in frequency domain, energy dissipation is usually treated independent of frequency and directional distribution of energy.

3.3 Diffraction and reflection

Diffraction and reflection processes due to the interaction of the waves with breakwaters can be treated usually as linear processes. They are strongly influenced by the directionality of the waves. The simulation method in frequency domain including the directional distribution is the tool for working out design diagrams. As an example diffraction and reflection at a semi-infinite total reflecting breakwater is discussed in the following.

Fig. 3.3 and 3.4 are results from calculation with regular waves (linear theory). Typically for the diffraction area in case of perpendicular wave approach (Fig. 3.3) is a relative wave height (diffraction coefficient K') of about K' ≈ 0.5 in the area of the 'geometric shadow line'. In the reflection area (example for oblique wave approach) immediately at the breakwater frontside the well known increase of wave heights up to K' ≈ 2.3 can be seen (Fig. 3.4).

Similar calculations with a directional spectrum of typical wind waves (JONSWAP, s_{max} = 10) are plotted in Fig. 3.5 and 3.6. In the diffraction area (Fig. 3.5) the relative wave heights in the area of the geometrical shadow line are increased to about K' ≈ 0.7 and generally the wave heights in the whole sheltered area behind the breakwater are significantly higher. However, in the reflection area with oblique wave approach (Fig. 3.6) the relative significant wave heights are clearly reduced. This is due to the fact that part of the wave energy (with direction beyond the direction of the breakwater axes) is not longer reflected, but diffracted.

For oblique wave approach of directional sea there is again the principle problem concerning the directional features of the input spectrum. Near the coastline, or e.g. for breakwater gaps, 'off-shore' directional components may not be present, so that the directional spectrum is not the standard case. It has to be made clear, whether this is already considered in the input design parameters and how it is to be implemented in the simulation method.

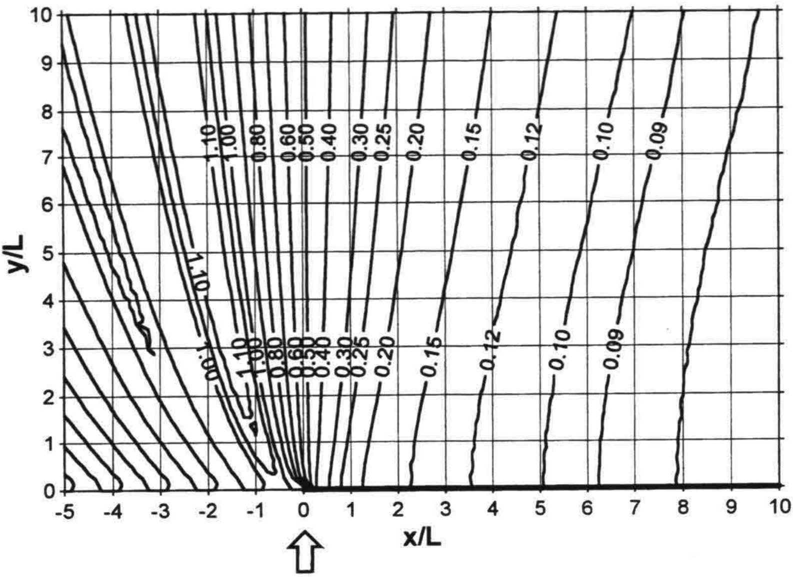


Fig. 3.3: Diffraction coefficients K' at a semi-infinit, total reflecting breakwater regular waves, perpendicular wave approach (Θ_0 = 90°)

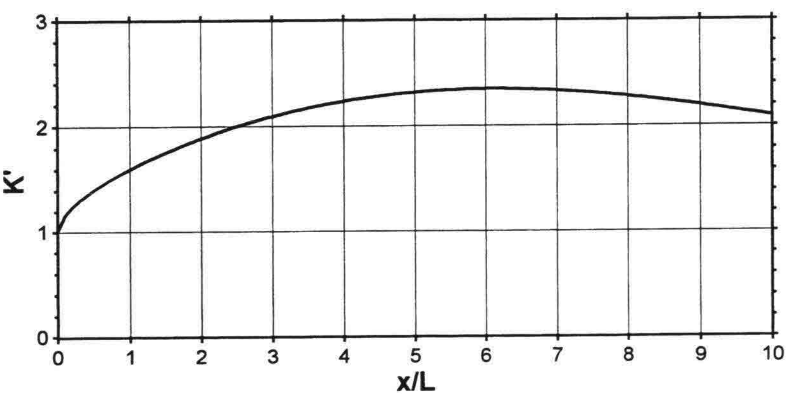


Fig. 3.4: Relative wave heights in the reflection area in front of a semi-infinit, total reflecting breakwater regular waves, oblique wave approach (Θ_0 = 20°)

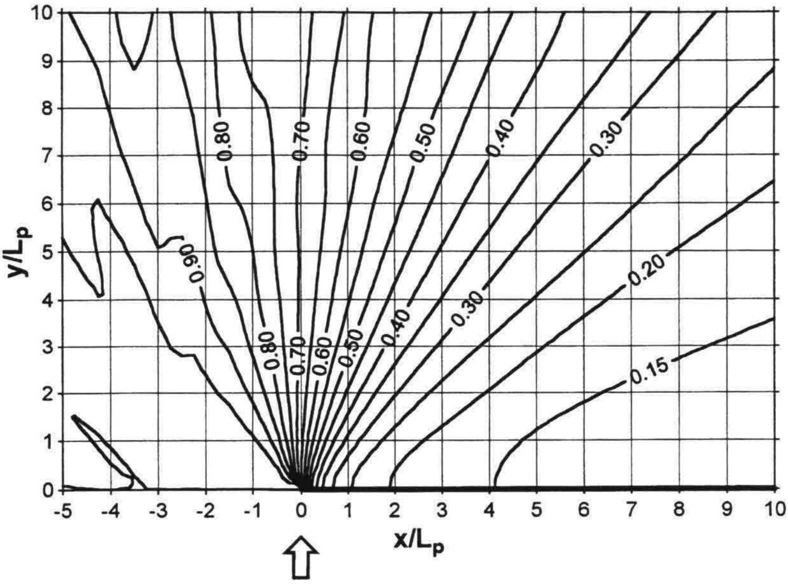


Fig. 3.5: Diffraction coefficients K' at a semi-infinit, total reflecting breakwater
directional spectrum JONSWAP (s_{max} = 10), (Θ_0 = 90°)

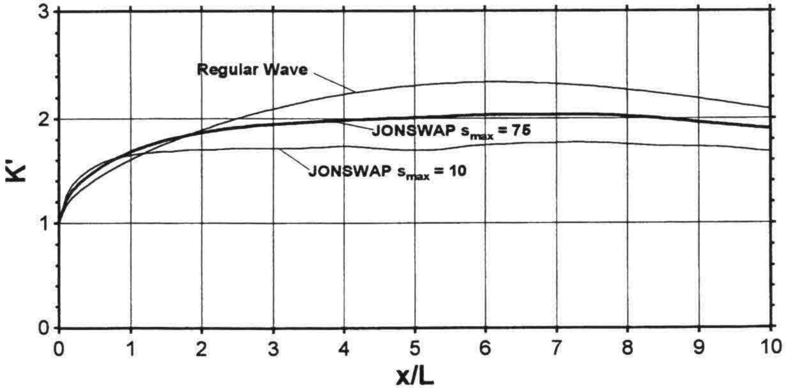


Fig. 3.6: Relative wave heights in the reflection area in front of a
semi-infinit, total reflecting breakwater
directional spectrum JONSWAP (s_{max} = 10 and 75), (Θ_0 = 20°)

3.4 Wave transmission

Wave transmission of irregular waves can be simulated with the transfer function method as long as no wave breaking occurs.

Fig. 3.7 gives an example of measured transfer functions for an immersed vertical thin wall with various degrees of obstruction (EGGERT, 1983).

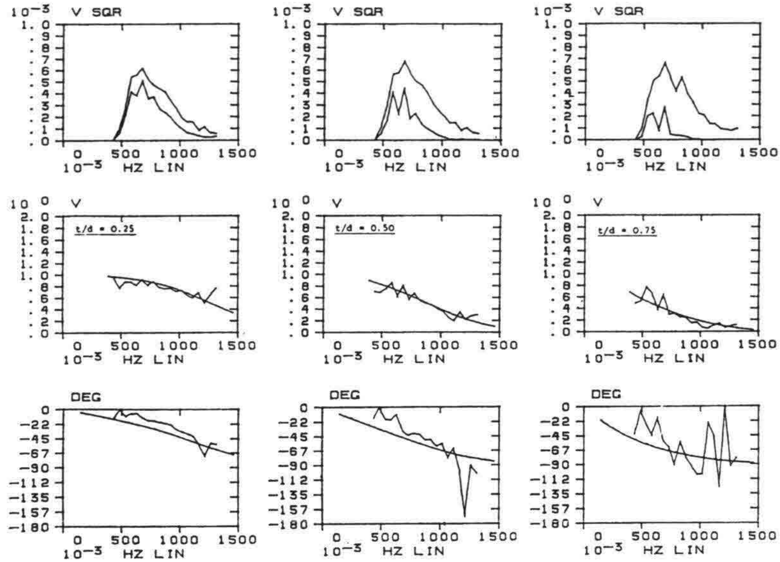
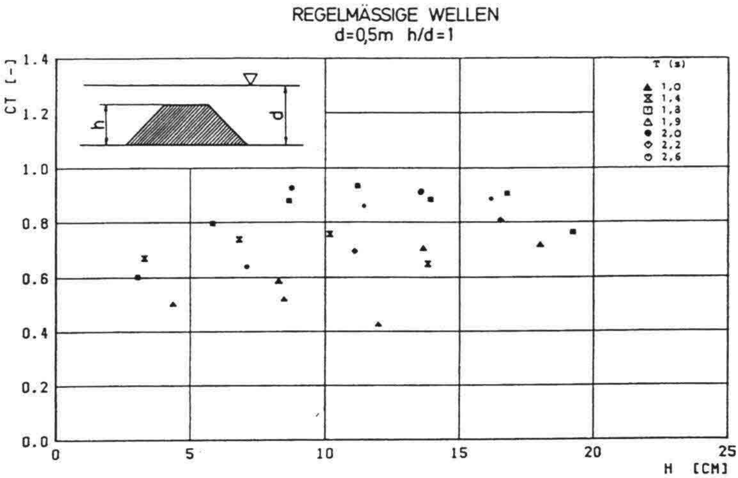


Fig. 3.7: Wave transmission at immersed vertical wall (EGGERT, 1983)
Transfer functions measured and from regular wave theory

In EGGERT, 1983 it is also worked out clearly, that for transmission calculations the representation of the sea state by a regular wave should make use of the mean period of the transmitted wave spectrum to get good results. He has given a diagram for estimating this significant period for standard spectra, but recommends the use of the transfer function method in general cases.

When transmission is controlled by the degree of wave breaking, even the simulation method in time domain may fail, although GODA, 1985 has given some examples where the representation of the irregular waves by a regular wave results in not too large errors. Own comparing investigations in a wave channel on submerged breakwaters (DAEMRICH/KAHLE/PARTENSCKY, 1985), however, highlight the problem of transferring results from regular wave investigations to irregular waves (Fig. 3.8). Hydraulic model tests are strongly recommended for this type of problems.



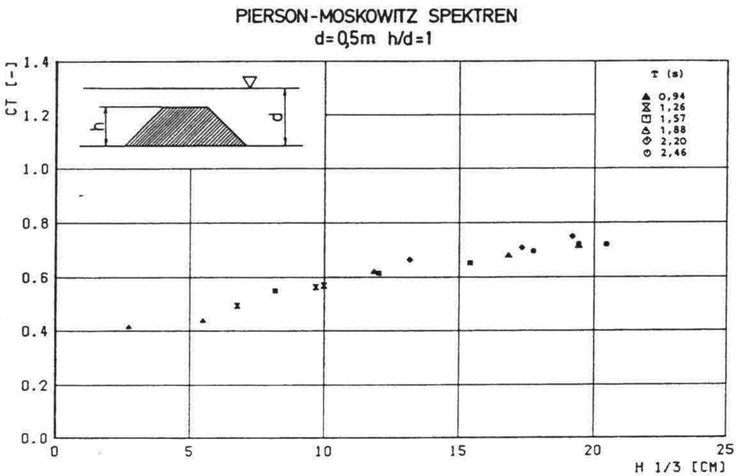


Fig. 3.8: Transmission at submerged breakwaters (DAEMRICH et al., 1985)
Comparison of results from hydraulic model tests in regular waves (top) and irregular waves (bottom)

The design method for submerged rubblemound breakwaters by VAN DER MEER / ANGREMONT, 1991 is based on the compilation of data from irregular wave tests in hydraulic models.

3.5 Wave run-up and overtopping

Wave run-up and overtopping of irregular waves has to be simulated in time domain.

For run-up at sloped sea dikes TAUTENHAIN, 1981 has estimated the influence of previous waves on the actual wave run-up and developed diagrams to calculate the overtopping quantities. Furthermore results for oblique wave approach have been worked out on the basis of hydraulic model tests (TAUTENHAIN et al., 1982).

GODA, 1985 worked out diagrams for overtopping of irregular waves at vertical structures and block mound seawalls, using the simulation method in time domain. The underlying results of regular wave tests are not given in the publication, however, some results of measurements indicate that large deviations can be expected when regular wave results are taken directly to represent irregular overtopping.

FRANCO/GERLONI/VAN DER MEER, 1994 have compiled design diagrams for vertical and composite breakwaters on the basis of measurements in irregular waves in hydraulic models.

For overtopping under oblique wave approach at vertical structures no general design diagrams or rules are available, although there is a strong influence. Hydraulic model tests have to be performed for each special case.

Some investigations on overtopping at vertical walls with directional spectra indicate that differences in results from long-crested and short-crested investigations are not severe.

3.6 Stability of rubble mound structures

Design of rubble mound structures according to the HUDSON formula is an example for the conflict arising from the use of regular wave results for design in irregular waves.

After publication of HUDSON's results from the laboratory investigations the Beach Erosion Board recommended in the Technical Report No. 4 (predecessor of the SHORE PROTECTION MANUAL) to use $H_{1/3}$ as design wave to be used with the stability coefficients measured in regular waves. This has been customary prior to HUDSON's investigations and arguments are, that normally a long period of destructive wave action is required before serious damage occurs. In later versions there was a slight modification in arguments, but $H_{1/3}$ was still recommended until in the latest version of 1984, when $H_{1/10}$ became the design recommendation. At the same time the stability coefficient was reduced, so that an increase of the stone weight by a factor of more than 3 was the result. The PIANC Working Group reports in 1992 that this change can be seen as altering the formula from a failure function to a design function with allowance for a margin of safety, but expects a review of the use of $H_{1/10}$ in the next up-date of the Shore Protection Manual.

In principle, due to the damage process depending on interdependence of waves and on possibly reshaping of the structure, only investigations in irregular waves are a suitable simulation method for this problem. There are hints from the literature (e.g. JOHNSON/MANSARD/PLOEG, 1978) that wave groupiness may have some influence. This might indicate that a representative wave height higher than $H_{1/3}$ is more significant for the damage process.

The formulae on the stability of conventional rock armour derived by VAN DER MEER (e.g. VAN DER MEER, 1988) are sole based on investigations with irregular waves. In the formulae $H_{2\%}$ is used as a representative wave parameter, related to $H_{1/3}$ by a factor 1.4 for standard cases.

3.7 Orbital velocities

Investigations on orbital velocities in a wave channel (DAEMRICH/EGGERT/ CORDES, 1982) are taken as an example to compare the various simulation methods and to demonstrate the influence of the shape irregularity.

The wave surface and the orbital velocity components were measured simultaneously. From the surface information, orbital velocity components were calculated for individual wave events, using simulation methods in time and frequency domain. Fig. 3.9 shows results from the simulation of horizontal velocity components as scatter plots.

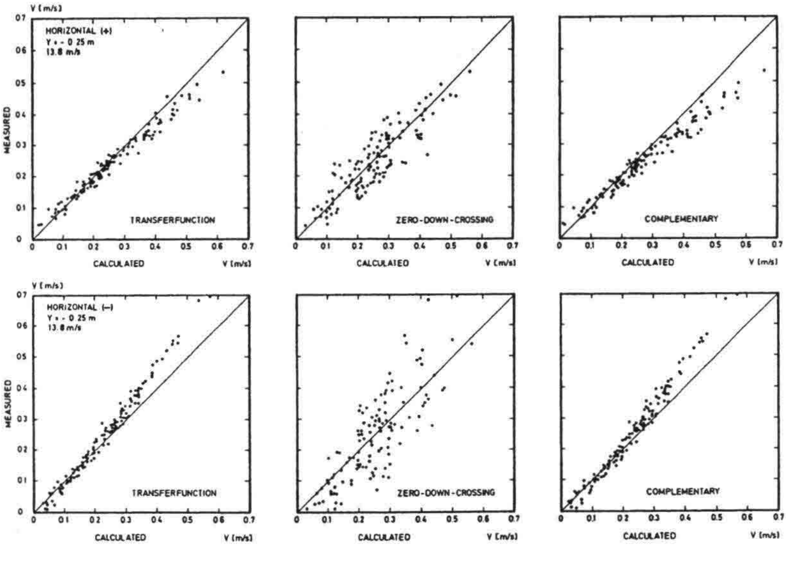


Fig. 3.9: Comparison between measured horizontal velocities and those calculated by different simulation methods (submerged depth 0.25m) JONSWAP spectrum U_{10} = 13.8 m/s, scale 1 :20

The scatter of the data from the simulation method in frequency domain (transfer function method) is relatively small. The scatter of the data from the simulation method in time domain with zero-crossing definition is markedly higher. This is to be attributed to the influence of the shape irregularity, as for the calculations all waves are assumed to be symmetric regular waves defined by H, T and the water depth only.

The results can be clearly improved, however, if more attention is paid to the individual wave form by using other definitions in time domain. The scatter in the results from calculations with 'halve wave parameters' (complementary method) are of the same order as the scatter from the transfer function method.

Despite of the relatively small scatter of the transfer function method and the complementary method the general trend of overprediction of crest velocities and underprediction of trough velocities is obvious. From investigations of WOLTERING, 1996 it is known that this trend is clearly due to the influence of the EULERian backflow in the hydraulic model, which was not considered at the time of this analysis.

Furthermore it has to be mentioned, that a linear superposition method of course must have limitations when the non-linearities grow stronger. Especially the crest velocities in high waves near the surface are clearly overpredicted by the transfer function method due to the non-realistic behaviour of the linear transfer function above the mean water level. Non-linear superposition methods (e.g. SAND/MANSARD 1986), superposition with varying (frequency dependent) reference still water level (DONELAN/ANCTIL/DOERING, 1992) or LAGRANGEian approach (WOLTERING, 1996) improve the results in details.

4 Summery and concluding remarks

Sea waves are irregular in contrast to regular long-crested waves of permanent form, where most of our knowledge, perception and wave theories are based on. The effects of the irregularity are due to directional spreading of the wave energy, irregularities in the sequence of wave heights and periods and the shape irregularity. Non-linearities are not an effect of the irregularity but are more complex in irregular waves.

Design methods can be based on the results of simulation methods in time or frequency domain, hydraulic model tests or measurements in nature. Selected examples are compiled in chapter 3. The methods are standard methods in general. In research more sophisticated methods might be applied for special problems and to account for non-linear and interaction processes. Fluctuations in the mass transport in irregular waves e.g. and the related problem of bound long waves is a specific problem which does not exist in regular waves and still requires a lot of research (GÖTSCHENBERG/DAEMRICH, 1990).

The design by representing irregular waves by means of a 'significant wave' is not impossible, but requires experience to avoid large errors. Regular waves with the height of $H_{1/3}$ are in fact not very often significantly representing the irregular wave action.

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Hydrodynamic Impact on Cliff Coast Areas of the Baltic Sea on the Basis of Directional Wave Measurements

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Abstract

Wide areas of the Baltic Sea coastline are characterised by cliffs and the alternation of active cliffs and adjacent accretion areas. Active cliffs are important for the sediment budget and the stability of the beaches. Driving forces for morphological changes and the movement of sediments are primarily the incoming hydrodynamic forces, namely waves, currents and changing water levels.

1 Introduction

The Institute for Hydraulic and Coastal Engineering of the University of Rostock together with the Geological Palaeontological Institute of the University of Kiel (GPI) and German governmental authorities (ALW Kiel, LANU Schleswig-Holstein) are running a joint research programme on the "Influence of Cliff Coast Erosion at the Baltic Sea on the Dynamics of Adjacent Shallow Water Areas". The basic objective of the research programme is to determine the relations between the hydrodynamic impact, the decline of the cliff and the development of the shore on the basis of field investigations and theoretical approaches.

Within the joint project the Institute for Hydraulic and Coastal Engineering is mainly working on the hydrodynamic impact into the project areas and on the hydrodynamic changes from deeper water towards the surf zone. The hydrodynamic impact into the project areas is beeing investigated on the basis of measurements, theoretical approaches and numerical as well as statistical simulation methods in connection with sedimentological and geological investigations and with detailed surveys of the project areas. The results of the wave climate analyses are input data for the sedimentological and geological investigations concerning the assessment of the behaviour of the cliffs and adjacent areas.

In this paper, results will be presented on the estimation of long - term hydrodynamic impact into the project areas on the basis of wave measurements and statistical approaches.

2 Methodology / Objectives

Since May 1996, directional wave measurements have been performed for the determination of the hydrodynamic impact into the project areas. The directional wave measurements are carried out in preselected project areas in about 10 m water depth with two Datawell Directional Waverider Buoys. The selected positions of the buoys within the project areas are representative for the complete project area respectively.

The main objective of the investigations of waves is the determination of the wave climate - in this association the term wave climate is used to express the wave conditions and its seasonal and local variability - in connection with actual water levels and local wind conditions. This contains, for example mean wave conditions, seasonal variability of waves and wave parameters within extreme conditions as well as probabilities of extreme conditions.

It is furthermore planned to analyse the transformation of the waves from deeper water into the surf zone on the basis of numerical modelling as one particular basis for the assessment of the cliff erosion and the sediment transport.

3 Project Areas

For the investigations carried out within the research programme three project areas were selected on the Baltic coast of Schleswig - Holstein. Main criterion for the selection of the project areas were, besides the different alignment of the cliffs to the main wave attack, results of geological and sedimentological

Fig. 1: Project areas at the Baltic coast of Schleswig - Holstein

investigations, which were carried out over the last years and which can be used for the investigation work within the research project. Considering these factors cliff coasts at Schönhagen, Brodten and Heiligenhafen (Fig. 1) were selected for the investigations.

Each of the three project areas consists of an erosive cliff with accretion areas in the adjacent sections of the coast. The mean decline rates of the erosive cliffs are high (up to 2.5m / year) in comparison with other cliffs in the western part of the Baltic Sea. The adjacent accretion areas and therefore the physiographic units are clearly defined and the input of sediment through rivers into the project areas can be excluded.

For the first part of the project, it was decided to concentrate the project activities on the project areas Heiligenhafen and Schönhagen.

4 Data Acquisition

4.1 Directional Wave Measurements

The directional wave measurements are performed with two Datawell Directional Waverider Buoys (pitch- and roll buoys). The diameter of a Directional Waverider Buoy is about 0.9 m and the weight about 250 kgs.

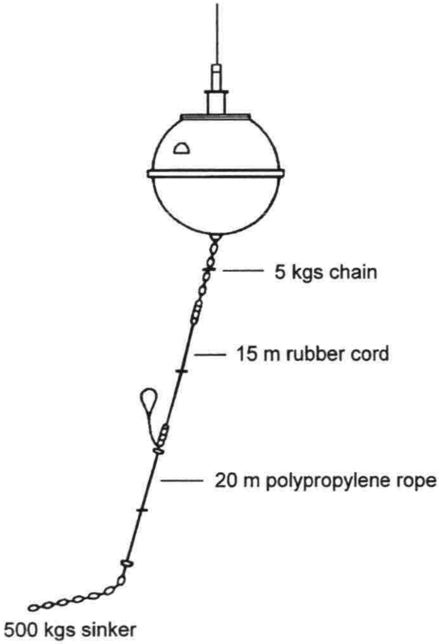

Abb. 2: Datawell Directional Waverider with flexible mooring system (according to DATAWELL 1995)

The buoys are moored with a flexible mooring system (Fig. 2) that includes a rubber cord with safety line, a polypropylene rope and a chain of about 500 kgs as a sinker. The flexibility of the mooring system ensures, that the buoy can optimally follow the waves on the water surface.

The measurements of wave heights and wave directions are based on measurements of accelerations in a vertical direction (heave) and in two horizontal directions which are perpendicular to each other. With simultaneous compass measurements of the direction north the horizontal data is converted into the directions N and W. By means of double integration of the accelerations, heave, translation W and translation N are determined respectively.

The sample frequency of the Directional Waverider Buoys is 1.28 Hz. 8 blocks of 200 s a time (≙ 256 samples) are measured and analysed for the calculation of one directional wave spectrum that represents the measured sea state of one measurement. The duration of one measurement is approx. 27 minutes.

The time histories of the measured components translation north, translation west and vertical heave are respectively developed in complex Fourier series. The Co- and Quad- spectra are calculated from the Fourier series. The energy density function, the mean wave directions and the directional spread are calculated from the Co- and Quad- spectra. The analysis of directional wave information is described in detail by Longuet Higgins et al. 1963.

An example of a typical directional wave spectrum is given in Fig. 3.

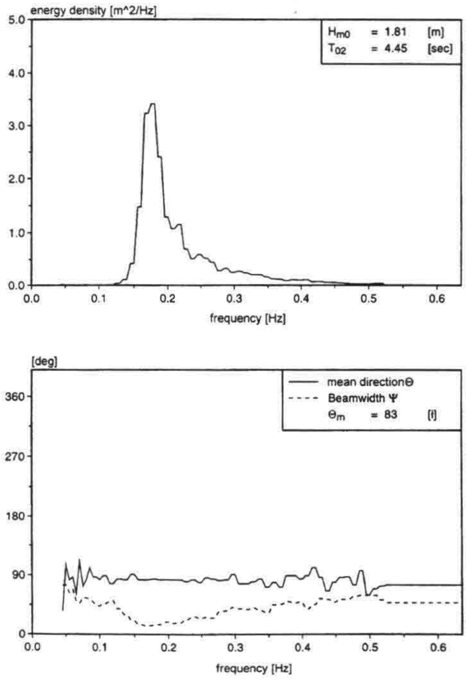

Fig. 3: Measured Directional Wave Spectra
Location Schönhagen

4.2 Data Analyses

The wave data is recorded every hour. The significant wave parameters of the wave spectrum are calculated from the energy density function and the wave directions of a measurement, i.e. H_{m0}, T_{02}, T_p and Θ_m and the significant wave parameters $H_{1/3}$, H_{max}, T_m, $T_{H1/3}$ from the measured time histories respectively. The processing of pitch- and roll buoy data in time and frequency domain is, for example, described in detail by SCHADE, 1991.

An example of the measured wave parameters is given in Fig. 4.

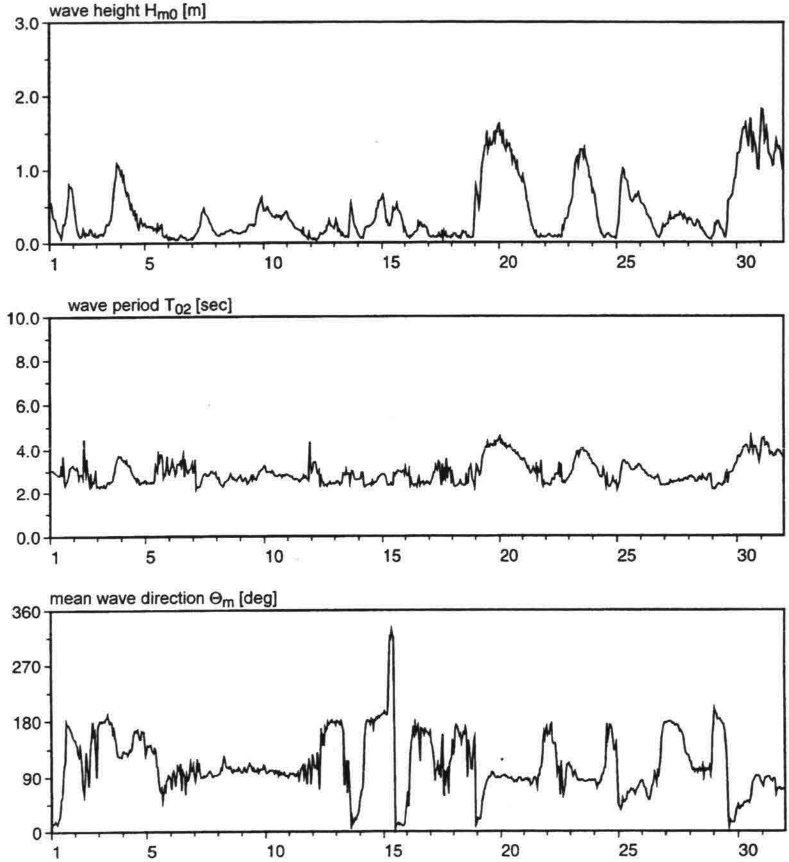

Fig. 4 : Measured wave parameters, Dec. 1996, Location Schönhagen

4.3 Wind- and Water level data

For the investigations of long - term sea state conditions in the project areas long - term wind measurements were used. The 'Deutscher Wetterdienst' (German Weather Service) provided long - term wind conditions measured at locations in the direct vicinity of the project areas. For the Schönhagen project area hourly wind measurements are available beginning in 1981 and for the Heiligenhafen project area, wind measurements beginning in 1957.

Water level data in 5-minute steps are also available at long - term water gauges directly in the project areas. The data was collected by 'Wasser- und Schiffahrtsamt' (WSA) Lübeck.

5 Estimation of Long-term Wave Climate

Especially for the assessment of morphological changes - long-shore sediment transport -, which takes place very slowly and over longer periods, a single record representing the waves of 1 hour only does not give the necessary information. In this case the information about the long-term behaviour of the waves is required. On the other hand it is very expensive and very time - consuming to measure the wave parameters over long periods. This makes it necessary to develop procedures to extend the time series of measured waves to longer periods. Since long term wind information is normally accessible for comparatively long periods, the extrapolation of the wave data can be performed on the basis of wind information.

5.1 Wind- Wave Correlations

For the statistical assessment of wave data and specially for the calculation of mean wave conditions and wave climate as well as extreme events it is necessary to use data from a complete and closed time series with constant resolution in time domain. Wave measurements are often incomplete due to various reasons such as ice coating of the ocean area or problems with radio transmission. Therefore it is necessary to complete the data gaps in the field wave data.

In principle there are many theoretical approaches available for the estimation of wave parameters from wind information. The appropriate approach normally depends on the available input information, namely the resolution of wind or related information in space and time, and also on the objective of the investigations. For the hindcast of wave parameters with a two dimensional numerical full spectral wave prediction model, detailed information (in space and time domain) about the atmospheric pressure field is necessary. The request for computational power is also extremely high. In return, detailed information on the spectral and the directional spreading of the wave energy distribution is available from the model. If in contrast to these sophisticated models, a very simple wave prediction model as for example a wave prediction model as described in the Shore Protection Manual (CERC 1984) is used, the demand on the input data is very low. In return, the quality of output data is comparatively poor and only significant wave heights and mean wave directions can be roughly estimated from the prediction model.

In addition to physically - based wave prediction models for the calculation of the relations between wave parameters and wind conditions, there are also statistical approaches. These statistical approaches are based on measurements. Measured wave parameters and simultaneously measured local wind parameters are necessary for the calculation of the relations. The advantage of statistical methods is that they are very fast in the calculation of the results and that they give comparatively accurate results for any of the correlated values. The limitation of statistical approaches is that they allow only the calculation (hindcast) of local wave parameters from local wind information. Statistical approaches can not normally be used to generalise/extend the results of the investigations in space.

In the context of the research project, a statistical approach was chosen for the calculations of the correlation between the wind and the waves. The procedure for the calculation is shown for the Schönhagen project area as an example.

The calculations are based on one year's measurements of wind and wave parameters. The total sample consists of about 3600 data sets of wind and wave measurements

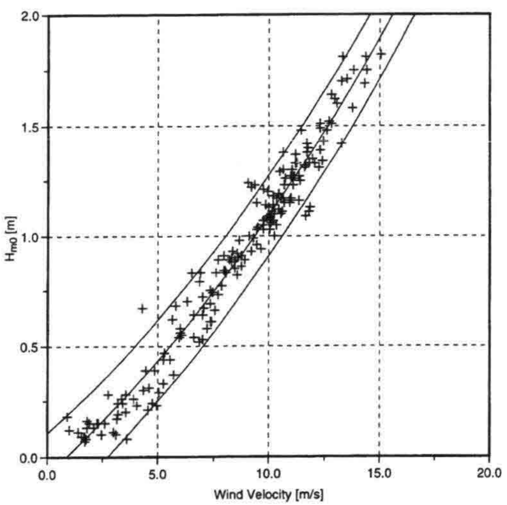

Fig. 5: Measured wave heights vs. measured wind velocities with correlation function and corresponding 90% confidence intervals. project area Schönhagen, wind direction: $40° \leq \Theta_{Wind} < 60°$

5.1.1 Wave Heights

For the calculation of the statistical interrelations between wind velocity and wave heights, the data was separated into classes of wind directions. For each class the wave heights are plotted against the wind velocity respectively. An example for the statistical relation is given in Fig. 5. The dependency of the wave heights on the wind velocities can be readily seen. The calculated correlation function and the corresponding 90% confidence intervals are also plotted in Fig. 5.

Detailed investigations showed that it is possible to improve the results of the regressions between the wind velocities and the corresponding wave heights. Within the improvement of the results, the statistical approach, the time differences between the wind and the wave measurements, the time over which the wind parameters are averaged and the class width of the wind directions were optimised. It was found, that a reliable criterion for the assessment of the goodness of the fitting is the mean absolute deviation between measured and calculated wave heights.

Statistical approach:
A variety of statistical approach functions (i.e. several types of power series and polynomal series of different degrees) were tested within the optimisation process. For the conditions at the Schönhagen project area a polynomal function of second degree gives the best results, which means the overall lowest differences between calculated and measured wave heights (lowest mean absolute deviation).

Influence of time differences:
Besides the absolute wind velocity and the direction of the wind, the duration of certain wind conditions has, for given local environment (water depth, morphological structures, etc.), the main influence on the development of the waves. For the calculation of the regression between wind velocities and corresponding wave heights, the influence of the duration and of the period which is significant for the development of the waves was analysed in detail. The investigations were performed with the actual wind parameters (i.e. the wind parameters measured at the same time as the wave parameters) and in hourly steps with the wind parameters measured before the wave measurements, as well as with wind parameters averaged over two, three and four hours. The investigations showed that correlations with wind parameters averaged over the last two hours before the wave measurement led to the best results.

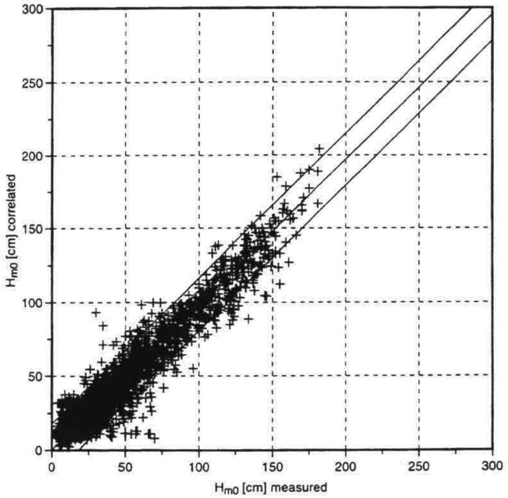

Fig. 6: Calculated and measured wave heights with linear regression function and 90% confidence intervals (project area Schönhagen)

Class width of wave directions:
For the optimisation of the

correlations, the class width was varied. An optimum class width was found at 30°.

As mentioned above, the mean deviation between measured and calculated wave heights may be taken as a reliable criterion for the assessment of the goodness of the correlations. The optimisation processes of the different main influences on the results of the correlations were comparatively successful. The absolute differences between measured and calculated wave heights are less than 7 cm on average and less than 15 cm for the 90% confidence intervals of the wave heights. The results are shown in Fig. 6.

5.1.2 Wave Directions

The calculation of the mean wave direction from measured wind data is performed on the basis of the same sample, which was used for the calculation of the wave heights. Comparative calculations showed that the results for the computation of the wave directions are not as strongly influenced by the different input wind parameters as the wave heights. In Fig. 7 the comparison of measured wind and wave directions are shown for the complete set of input data. It can be readily seen, that there is no direct dependency between the measured wind and wave directions. The resulting wave directions are completely different to the wind directions, especially for wind blowing from westerly directions (wind blowing from land to sea). They also diverge in two different directions.

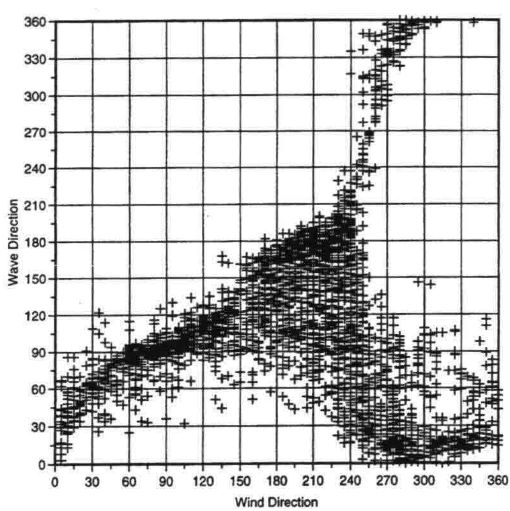

Fig. 7: Measured wind and wave directions project area Schönhagen (total sample, 3600 data sets)

Looking further into the details, it was found that the relations between the wind directions and the resulting wave directions are not as chaotic as they seem to be.

For wind velocities exceeding 7.5 m/s, a strong relation was derived from the data (cf. Fig. 8). The waves come more or less from the same direction as the wind.

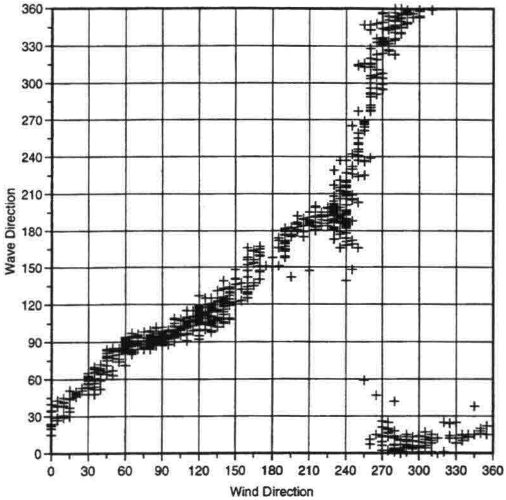

Fig. 8: Measured wind and wave directions project area Schönhagen (wind velocity $U_{Wind} > 7.5$ m/s)

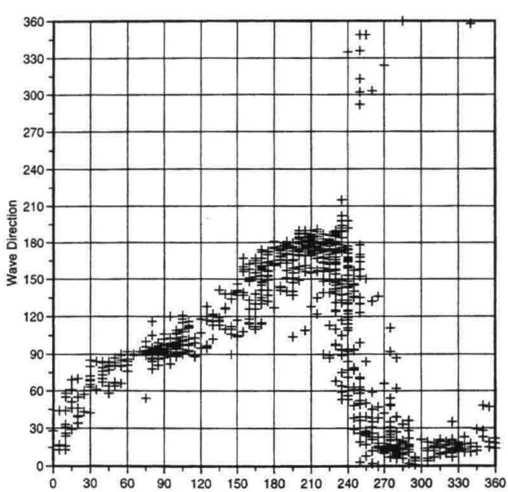

Fig. 9: Measured wind and wave directions project area Schönhagen (wind velocity 7.5 m/s > $U_{Wind} > 5.0$ m/s)

For wind velocities between 5.0 m/s and 7.5 m/s, non - linear relations were also derived from the measured data (Fig. 9). Compared with the data exceeding 7.5 m/s wind velocity, the bandwidth of the results is higher, but for the calculation of the wave direction from the wind direction, the results are reasonable.

For wind velocities below 5.0 m/s the relations are not as clear as for the previous examples (cf. Fig. 10). The spreading of the results is comparatively large, especially for wind coming from westerly directions. But for such climatic conditions, the wave heights and therefore the wave energy approaching the coast is very low and the wave heights are only in the range of 0.10 m to 0.25 m. Therefore the correlations can be used to obtain a completed series of wave conditions.

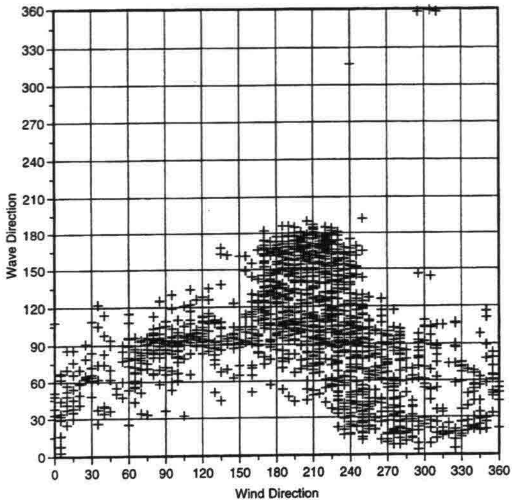

Fig. 10: Measured wind and wave directions
project area Schönhagen
(wind velocity $U_{Wind} < 5.0$ m/s)

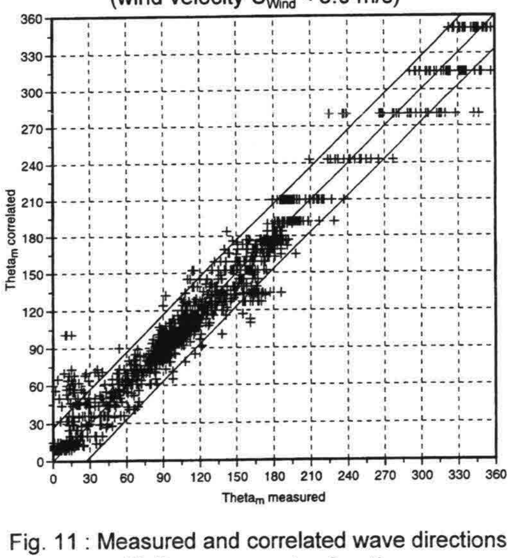

Fig. 11 : Measured and correlated wave directions
with linear regression function
and 90% confidence intervals
project area Schönhagen

For the calculation of the wave direction from the wind direction, non - linear functions defined in sections were found depending on both the wind direction (Θ_{Wind}) and the wind velocity (U_{Wind}). The results of the wind wave correlations are shown in Fig. 11. The deviation between calculated and measured wave direction is (on average) in the range of ± 20° for the complete range of data and ± 10° on average for wave conditions with significant wave heights exceeding 20 cm respectively. The mean wave direction is represented very well by the developed wind ⇒ wave correlations.

5.1.3 Wave Periods

The calculation of the wave periods from wind measurements is based on the relationship between wave heights and wave periods (Fig. 12). The correlation between wave heights and wave periods is almost linear for wave heights exceeding $H_{m0} = 0.25$ m. For wave heights below $H_{m0} = 0.25$ m, no direct correlation can be found. The great bandwidth of possible wave periods for comparatively low wave heights can be explained by swell effects in the project area. The investigations on the effects of swell on the wind wave correlation (especially wave periods and directions) have not yet been finally completed.

However, for the calculation of the wave periods from the correlated wave heights the relations derived from these investigations can be used. The mean deviation between calculated and measured wave periods are comparatively small. They are (on average) in the range of 0.2 s.

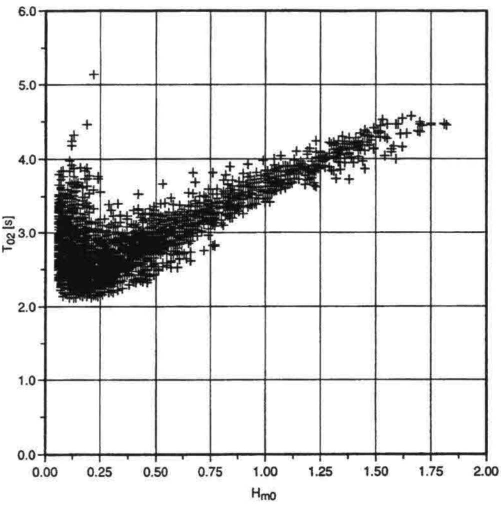

Fig. 12 : Measured wave heights and measured wave periods project area Schönhagen

The calculation for one month is exemplarily presented in Fig. 13 to assess the quality of the calculation of wave parameters from measured wind parameters. It can readily be seen, that the overall results are very reasonable. Not only are the mean values represented well, but also the course of the parameters. The results of the hindcast values based on local wind measurements are reliable even when the wave heights and the wave directions change comparatively fast. To summarize it can be concluded that by means of the investigations of the wind wave correlation, a hindcast tool for wave parameters (wave height, direction and period) for the project areas has been developed and that this tool is operational for further investigations.

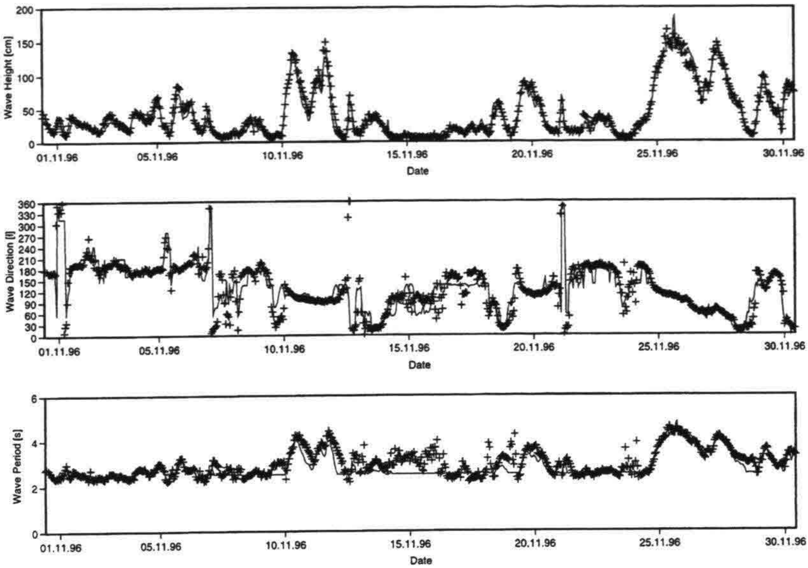

Fig. 13: Measured and calculated wave data (project area Schönhagen, November 1996)
+ : measured wave data
solid line: calculated wave data

5.2 Extrapolation of Wave Data

Based on the investigations of the correlations between the local wind parameters and the measured wave parameters, the wave data base can be extended to longer periods. For the Schönhagen project area, wind data measured at the same location is available from Aug. 1981 onwards and for the Heiligenhafen project area from July 1957 onwards.

With these two components, local wind information and wind wave correlation function, the wave parameters for the two project areas are principally available. An example for a wave hindcast is given in Fig. 14.

Especially for the calculation of the hydrodynamic impact into the project areas it is necessary to know whether there are other climatic conditions (e.g. ice conditions in the project area) that actively prevent / suppress the development of waves. The development of waves was assessed on the basis of ice coverage and ice thickness observations. Due to limited space these investigations cannot be described in detail.

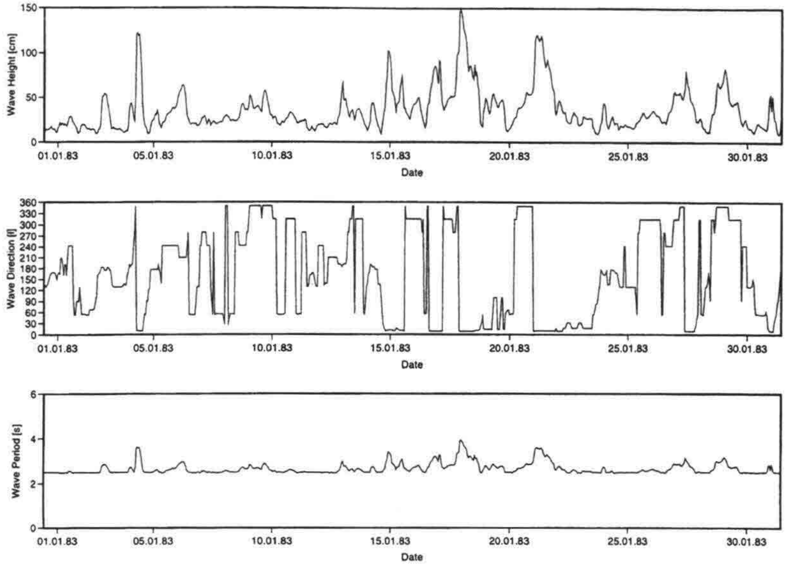

Fig. 14 : Wave parameters January 1983 project area Schönhagen, example from hindcast data

6 Hydrodynamic Impact

As an example for the use of the extrapolated wave data within the scope of our joint research project, the wave energy flux for the project area Schönhagen was calculated for a period of 15 years from Oct. 1981 to Sep. 1996. In Fig. 15 the monthly cumulated total wave energy flux, which characterises the hydrodynamic impact into the project areas, is plotted over this period.

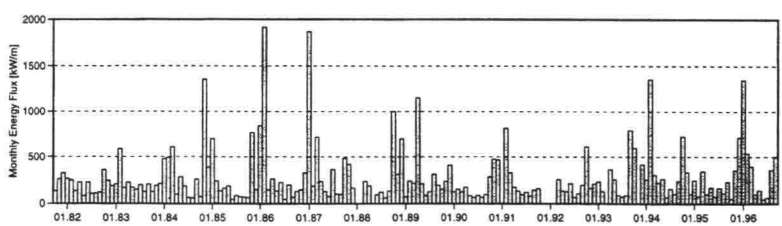

Fig. 15 : Monthly cumulated wave energy flux, Project area Schönhagen

The local wave energy flux will be used to estimate the local sediment transport capacity and also to assess the cliff coast decline and the development of the adjacent stretches in the project areas.

The mean wave conditions, the seasonal variability of the waves and also the assessment of the influence of extreme wave conditions will be investigated on the basis of the extrapolated wave data.

7 Conclusions

Within the scope of a seminar paper, it is not possible to deal in depth with all the various aspects of the assessment of the hydrodynamic impact into project areas. Above all results concerning the influence of increased water levels caused by strong winds or the local variability of the wave conditions and the importance of extreme events on the morphological development of the project areas had to be left out of this paper.

Within the scope of a joint research programme, directional wave measurements were carried out in two project areas over about 1 - 2 years. On the basis of these short - term wave measurements and local wind measurements wind $\Rightarrow$ wave correlations for the significant wave parameters H_{m0}, Θ_m and T_{02} were developed. Calculated and measured wave parameters are (on average) nearly equal. The differences between measured wave parameters and the calculated values are comparatively small. The mean deviation / spreading of the results of the complete data basis is about 7 cm for the wave heights, about 0.2 s for the wave periods and about 20° for the wave directions. For wave heights below 0.2 m, the spreading of the results for the wave directions decreased to about 10°. The spreading of the results for the wave heights and the wave periods was not changing significantly.

On the basis of the wind $\Rightarrow$ wave correlations the wave data base was extended to longer periods for the project areas. The local wave energy flux was calculated for one project area for the period of 15 years from Oct. 1981 to Sep. 1996 as the basis for the investigations of sediment transport processes and the sedimentological development of the area.

8 Acknowledgements

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Measurement of Directional Waves by Wave Gauge Array and Buoy

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ABSTRACT

For the operational oceanographical observation, the development of the pile station equipped with wave gauge array and the data buoy are introduced. Some important key points in the design of the systems are reported.

For the selection of a proper method for the routine directional wave analysis, a series numerical simulation have been carried out. By considering the stability, accurate wave direction prediction and time saving, Finite Fourier Series Method is suggested for the operational field wave analysis.

1. Introduction

Adequate oceanographical data is essential to the weather forecasting, the offshore structures design and the coastal zone management. An oceanographical observation network around Taiwan has been planned in order to fulfill the needs of related data. The network consists of stations located on the shore, in the shallow water as well as in the deep sea. In order to support the operational observation, pile station and data buoy systems have been developed. For the sake of the calibration of the numerical wave forecasting model, systems are designed to communicate with land-based stations by radio or satellites in real-time. In this report, the design of pile station and data buoy systems are introduced.

For the oceanographical observations, wave is, among others, the most predominate and complicated one. While other elements are obtained simply from the direct sensor reading, wave is presented by the characteristic values, which are obtained from the analysis of the measured water level time series. Directional spectra are the fundamental presentation of ocean waves. Several methods for estimating directional wave spectrum are investigated. The intercomparison between each method including FFSM, MLM, BAM, and MEM is presented in this report.

2. Field Observation

The measurement of ocean waves had long been of particular interest, as wave data and the understanding of wave phenomena are essential to ocean engineering. For routine, reliable and continuous observation, measurement of the wave can be, among others, recognized as data buoy and pile station or platform equipped with gauge array.

2-1. Wave Gauge Array

For the directional wave measurement, the pile station or platform equipped with gauge array are among the earliest in situ systems, and have the advantages of ruggedness. Several types of sensors, such as pressure, electrical, float and accoustic gauge can be used for the measurement. In the present study, acoustic wave gauge had been chosen for its relative ease of recovery and servicing. The acoustic gauge features no physical link to water surface. It beams a signal downward. The time required for the trip of signal serves to locate the water surface. By considering the design of the wave gauge array, the geometreical spacing effects the performance largly. For optimum performance, there should be as many nonredundant spatial pairs between each wave gauge as possible. By which, it is suggested that, there should not be any pairs of gauges toward the same direction, while the distances of each pair should not be identical. Fig.(1) shows the structure of the pile station. Two kinds of gauge array with 5 or 4 gauges are illustrated in Fig.(2) & Fig.(3), respectively.

The measurements are carried out hourly. The data are transmitted by radio to nearby shore station after each measurement, and then transfered to the laboratory at National Cheng Kung Univ. by telephone network, where the data are quality checked and stored. To safeguard against the loss of data due to transmission difficulties, data are stored in a hard disk on the pile, too. In the laboratory, data are quality checked to ensure the veracity before it is sent to the databank. The data quality control consists of automatical and manual procedures. As mentioned by NDBC, the automatical data quality control excutes in conformity with three guide-lines including limitation of the sensors, corelation of physical properties and continuity of the data. The data flow is shown in Fig.(4).

2-2. Data Buoy

In the deep sea or where the pile station is not available, data buoy is the most frequent applied tool, by considering real-time data transmission. A data buoy operation system is composed of several sub-systems as shown in the block-diagram in Fig.(5). For the wave measurement, the data buoy should have good wave following capability, thus disc type buoy was chosen. For moderate payload and convenience of land transportation, the present data buoy has a diameter of 2.5m. A drogue element suspended below the buoy is designed to avoid turnover. The outlay of data buoy is illustrated in Fig.(6). The vertical motion of the buoy is measured by an accelerometer located at the center of the gravity of the buoy. Wave height is obtained by the double integration of the acceleration with the time. The wave directionality is measured by two inclinometers built in the buoy due to the fact that the buoy inclines toward the direction coincides with the wave propagating direction. The inclination of the buoy , i.e. the pitch and roll, should be measured together with a 3 axis fluxgate compass to give the buoy real orientation. To avoid aliasing, signals are lowpass filtered with a Butterworth filter to eliminate the noise. The data-flow is the same as the pile station system in Fig.(4).

3. Data Analysis

Ocean waves are commonly represented by the statistical values or the spectra density. By considering the directionality of the waves, directional spectra analysis should be applied.

3-1. Fundamental Equations for Directional Spectrum Analysis

Directional spectrum can be derived by cross power spectrum which is the Fourier transformation of the covariance function of any two measured wave properties. The general relationship between the cross power spectrum and the directional wave spectrum introduced by Isobe et al.(1984) is adopted in the present study:

$$\begin{aligned}\Phi_{mn}(f) = \int_0^{2\pi} H_m(f,\theta)\cdot\overline{H_n}(f,\theta)\cdot\{&\mathrm{Cos}[k\cdot(x_{mn}\mathrm{Cos}\theta + y_{mn}\mathrm{Sin}\theta)] \\ &- i\,\mathrm{Sin}[k(x_{mn}\mathrm{Cos}\theta + y_{mn}\mathrm{Sin}\theta)]\}\cdot S(f,\theta)d\theta\end{aligned} \tag{1}$$

where f is the wave frequency; k is the wave number which could be obtained by the dispersion relationship from the frequency f; $\Phi_{mn}(f)$ is the cross power spectrum of the m-th and the n-th wave property. $S(f,\theta)$ is the directional wave spectrum; X_{mn} and Y_{mn} are the location vectors ; $H_m(f,\theta)$ is the transfer function which can be exprssed as:

$$H(f,\theta) = h(f)\mathrm{Cos}^{\alpha}\theta\cdot\mathrm{Sin}^{\beta}\theta \tag{2}$$

where h(f) and the parameters α and β are specified for each measured quantities and can be derived by linear impulse-response method.

The directional spectrum is often expressed as the product of the frequency spectrum *S(f)* and the directional spreading function $D(\theta|f)$.

$$S(f,\theta) = S(f)\cdot D(\theta|f) \tag{3}$$

The directional spectrum takes non-negative values and satisfies the following relationship:

$$\int_0^{2\pi} S(f,\theta)d\theta = S(f) \tag{4}$$

Substitution of the above equation into Eq.(3) yields

$$\int_0^{2\pi} D(\theta|f) = 1 \tag{5}$$

For the wave gauge array, each gauge pair forms a cross spectrum. N gauges form N(N-1)/2 cross spectra. By using Eqn.(1), N(N-1)/2 equations can be established, accordingly. The more the equations are, the more accurate result is gained. However, due to the fact that the number of the gauges is limited, the analysis result is always an approximation.

For the data buoy, the location vectors both in x and y direction in Eqn.(1) equal to zero, as the buoy measures heave, pitch and roll at a single point. Though the buoy moves with the wave orbital motion in the Lagrangian way, the analysis treats the signal in Eularian way due to the fact that the length of the orbital axis is neglegible in comparison with the wavelenght. Three cross spectra associated with three auto spectra from heave, pitch and roll signals can be formed. The directional spectrum can be obtained by solving Eqn.(1) associated with Eqn.(3).

3-2. Methods for Estimating Directional Spreading

In the Eqn.(3), the directional spreading $D(\theta|f)$ can be determined by various methods. According to Benoit(1992), methods commomly used for estimating directional spectrum are as following: Finite Fourier Series Method(FFSM), Fit to Normal Distribution Method(NDM), Fit to Bimodal Normal Distribution Method(2NDM), Long-Hasselmann Method(LHM), Maximum Likelihood Method(MLM), Iterative Maximum Likelihood Method(IMLM), Convolutive Maximum Likelihood Method(CMLM), Maximum Entropy Method(MEM) and Bayesian Approach Method(BAM). It should be noted that different methods give different results. While the applications of the data are different, e.q. for wave research or for engineering design, and due to the fact that each method has its constraint and limitation, the selection of the methods depends largely on the application of the data. Four methods ,i.e. FFSM, MLM, MEM, BAM, have been worked out and discussed in present study.

3-3. Comparison by Numerical Simulation

To identify the features of each method and as a result to help in selecting a proper method for routine observation, a seires of numerical simulations have been performed. The procedures of the simulation are as following: first, set a target directional spectrum; second, construct cross spectra from the target spectrum by using Eqn.(1); third, use these cross spectra as input and analyze the directional spectra by the four methods mentioned in previous section and finally, compare the directional spectra analyzed by the four methods with the target value. JONSWAP spectrum modal and Goda's directional spreading function were used to form the target directional spectrum.

A series of numerical experiments with various combination of directional spreading parameter S in Goda's formula and the spectra peakness parameter γ in JONSWAP spectra were carried out. Fig.(7) is a 3 dimensional plot in one case in which S=10 and γ=3. From the results, it can be seen that the FFSM and the MLM underpredict the maximum peak and overpredict the spreading. The MEM a little overpredict the maximum peak , and the BAM is very accurate in the case. For the predominate wave direction, however, all the methods perform well.

3-4. Discussion

The directional spectrum estimator for routine operational observation should have the features of time-saving, stable and be able to represent the essential characteristics of the field wave. To obtain an overview of the features of each method, five criteria including the time-saveing consideration, robustness, convergence, directional spreading estimation and predominate wave direction estimation are introduced. The features of four methods related to the criteria are listed in table(1). On the time-saving consideration, the FFSM is easy to implement and does not require lengthy computations such as matrix inversions or eigenvector calculations common to other methods. It is the most efficeint method. Both MEM and BAM need complicated iteration calculation. The iterative schems comsume lenghthy time. The robustness represents the ablity of methods for the processing data with noise. From the viewpoint of robustness, performance of MEM and BAM diminish with increasing noise. MLM may lead to bias estimation in some cases. FFSM gives the most robust results. Convergence is the problem in the iteration procedures. In some conditions, MEM and BAM give no results due to the divergence of the scheme during iteration computation. FFSM and MLM have no such problems as they do not need complicated iterations.

For the directional spreading estimation, MEM and BAM give the best results. They can always give accurate result for all cases, providing they don't diverge in the iteration procedure. MLM and FFSM usually underpredict the peak energy especially when the spreading parameter S is large. To solve the problem, more gauges are needed. However, all the four methods give good information to the predominate wave direction.

In summarized, in order to ensure the results free from the risks of loss of wave direction information in the routine operational observation, the robustness and the convergence are the most important criteria for the selection of the estimator. The computation time should also be considered as the time is very limited in hourly routine observation. The predominate wave direction is needed to present the sea status. By considering all the criteria as a whole, FFSM is suggested due to its stable, time saving and accurate wave direction estimating characteristics. However, one should keep in mind that the directional spreading given by FFSM would not be accurate enough, especially in the swell when the parameter S is large. To solve the problem, more gauges are needed in the gauge array.

4. Conclusion

Two wave measuring systems, wave gauge array and data buoy, are developed. Currently, three pile stations equipped with wave gauge array have been set-up in Taichung, Taishi and Kaoshiung in 1991, 1992 and 1996, respectively. According to the six years' experiences in the field observation, the wave gauge array has been proven to be a good method for the long term observation. The system is very stable and easy for maintenance. Two data buoys have been launched in Hwalien and Hsinchu, respectively, in June 1997. A comparison study shows that the directional spectra measured by data buoy are in conformity with that obtained by wave gauge array quite well.

For the directional wave analysis, four methods, i.e. FFSM, MLM, MEM and BAM, were examinated. It has been found that each method has its constrait and limitations. For the long term observation, stablity and the correct estimation of wave direction are essential. And time saving should also been considered. From these viewpoints, FFSM is suggested for the routine observation in the present study.

5. Acknowledgment

The authors gratefully thanks the National Science Council, the Central Weather Bureau and the Water Resource Bureau for their long years financial support in the development of the systems.

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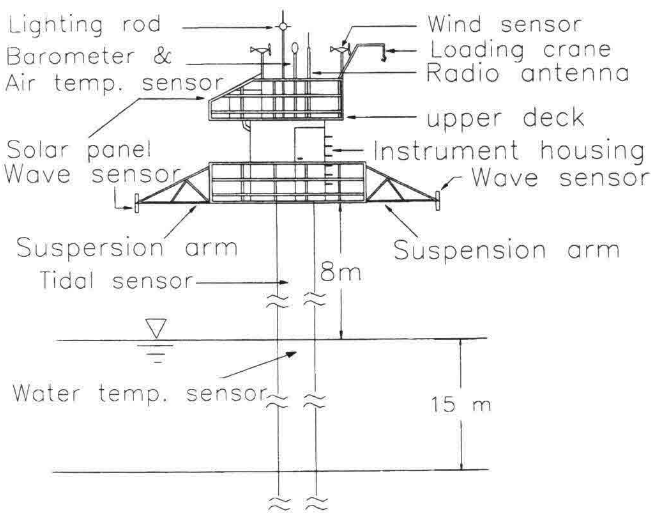

Fig.(1) The outlay of Pile Station

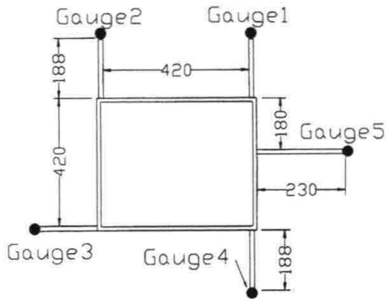

Fig.(2) The Arrangement of 5 Wave Gauges (Unit : cm)

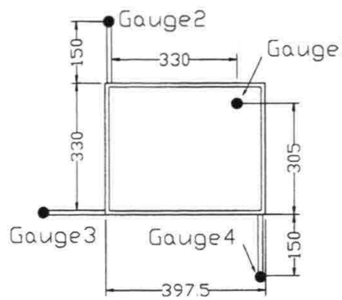

Fig.(3) The Arrangement of 4 Wave Gauges (Unit : cm)

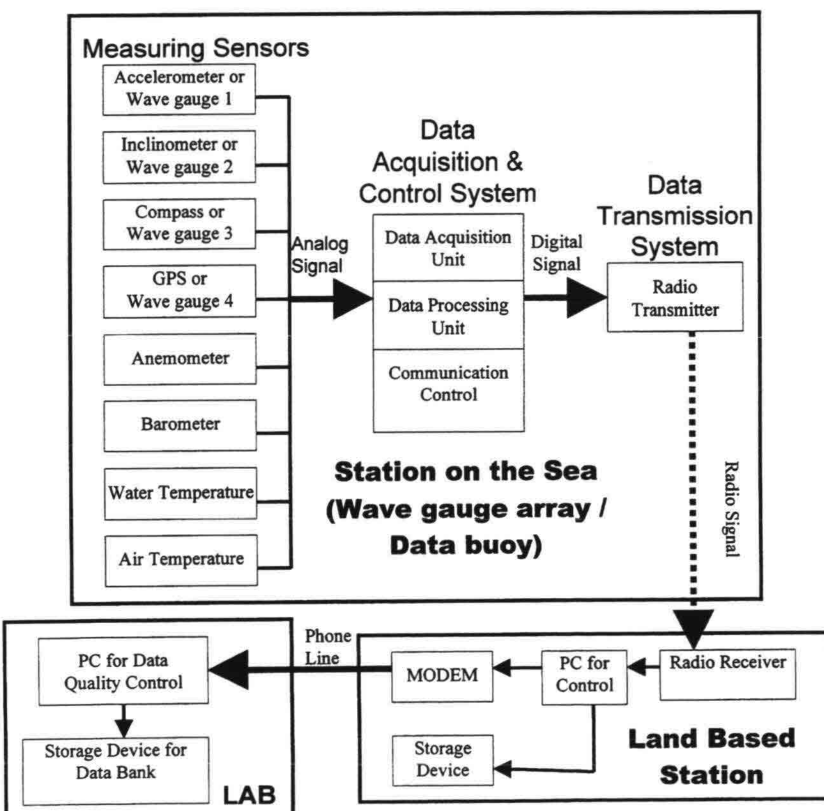

Fig.(4) Data flow of the Observation System

Data Buoy Operation System

*Hull & Mooring System	* Field System	*Land Based System	* Technical Support
*Structure Design	*Measuring sensors	*Data Receiving	* Launch
*Payload Design	*Data Acquisition	*Data Processing	* Retrieve
*Mooring & Anchor	*Data Analysis	*Data Analysis	* Maintenance
*Anti-Corrosive Process	*Data Storage	*Data Management	* Data Quality Control
*Safety Design	*Data Transmission	*Data Storage	
	*Power Supply	*Data Transmission	
	*Navigational Alarm		

Fig.(5) Components of Data Buoy Operation System

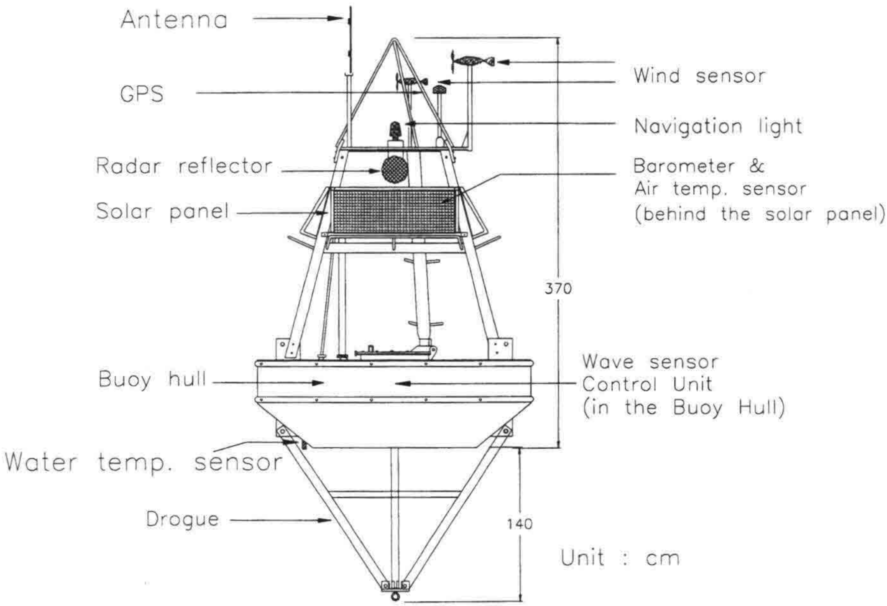

Fig.(6) The Outlay of Data Buoy

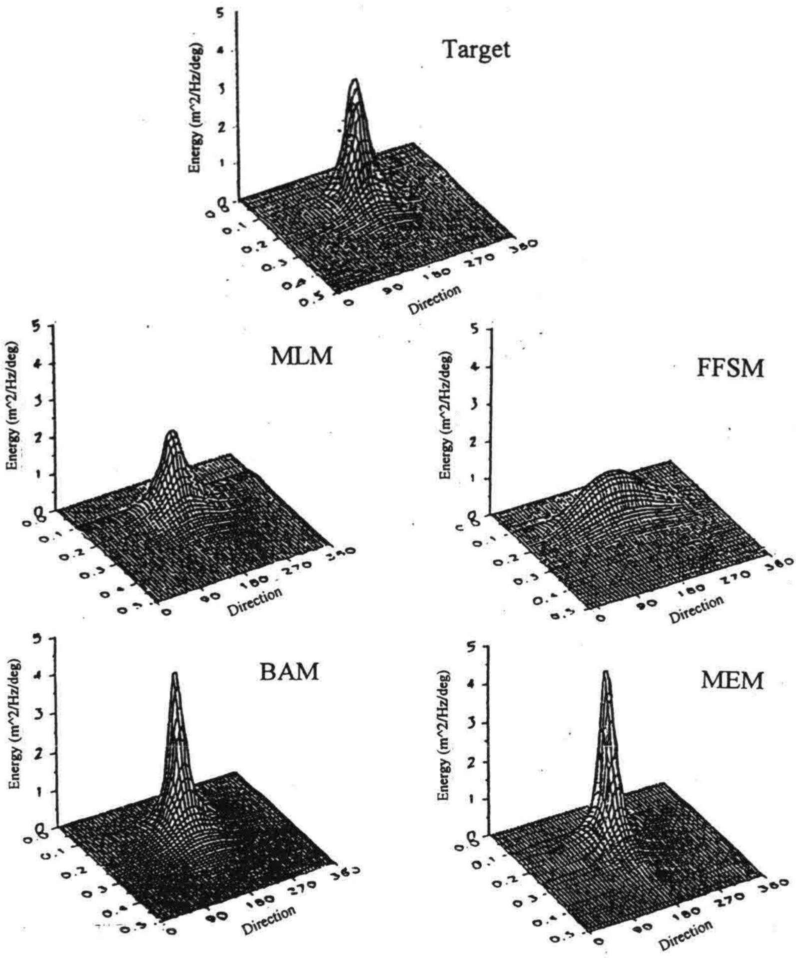

Fig.(7) Comparison of Results from Each Estimator

Criteria \ Methods	FFSM	MLM	MEM	BAM
Time saving	Good	Moderate	Poor	Poor
Robustness	Good	Moderate	Moderate	Moderate
Numerical Convergence	Good	Good	Poor	Moderate
Directional Spreading Estimation	Poor	Moderate	Good	Good
Predominate Direction Estimation	Good	Good	Good	Good

Table.(1) Features of Each Methods

Topic II

Field Investigations

Assessment of Results and Wave Prediction

Chairman: Karl-Friedrich Daemrich

Accuracy Analysis of SURFER® Contouring of Bar-type Coastal Topography

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Abstract

Computer-aided contouring of coastal bathymetry has become more popular than ever in the recent decade. However, selecting an appropriate software to perform contour-mapping may be as important as the bathymetric surveying in creating realistic contours. In this paper, resultant difference caused by different interpolation methods in the commercial mapping software, Surfer®, has been presented, and the corresponding effect of the sampling interval in a coast water prescribed by a three-dimension bathymetry equation with sand bars has further been studied.

About three hundred sixty resultant data sets have been created by the three interpolation algorithms in Surfer® from 120 sampling data sets directly computed from the given equation by varying the surveying interval between the running lines perpendicular to the coastline and the sampling distance in each running line. In each resultant data set, the water depths of 25 random-selected points were calculated by different interpolation algorithms and compared with the depth given by the bathymetry equation. The resultant average and standard deviation were then computed and plotted corresponding to the sampling distances and surveying intervals for comparison.

Results show that the Surfer®-MinCurv is better than Surfer®- InvDist and Surfer®-Kriging in contouring the topography given by a continuous function. Alongshore and on-offshore sampling spacing between 50 to 100m might be a reasonable number for a typical coastal with sand bars, but should be applied with caution. It is also found that unexpected error may occur when the sampling spaces is too small, e.g. about 30m in the study. Since interpolation algorithms may be sensitive to the surveying pattern such as the sampling spacing as well as the running direction, a software should be used with caution and some pre-study on the characteristics of the field topography such as the size of the alongshore bar should be done before a surveying pattern is scheduled.

1. Introduction

Water depth surveying and contour mapping of coastal water is one of the most important and basic works in almost all coastal engineering problems such as planning and design as well as coastal processes. A realistic and accurate bathymetry map strongly depends on the surveying procedure and great care on the contour mapping. Finer surveying interval and sampling distance are believed to yield more accurate result. However, a rough surveying scheme may result from the limitation of budget, man power, and wave state, etc. On the other hand, man-made contour mapping is extremely time- consuming and has almost been replaced by computer-aided mapping method. Consequently, it is important to know how the accuracy of the resultant map is or how it is affected by different interpolation algorithms used by different softwares while the surveying interval and the sampling distance are varying.

There are two major approaches to draw contour lines in most commercial packages. One of them is the gridding method [2, 3] in which some interpolation algorithms, such as the inverse distance, kriging, minimum curvature, and polynomial etc. have been applied to calculate the depths of the grid mesh points. Then the contour lines between the grid points can be determined by linear interpolation. Another approach is the Triangulation Irregular Network (TIN) [2, 3] which uses all the observed depth data points to form a unique triangular network from the theory of Voronoi Diagram and Delaunay triangle [2]. The contour lines are then determined by linearally interpolating between the data points. In this paper, the commercial software, Surfer®, has been applied. Three gridding algorithms in Surfer®, such as inverse distance, kriging, and minimum curvature, have been used [1].

Because of the difficulty of verifying the accuracy for a resultant bathymetric map from a set of measuring water depths, a theoretical topography containing alongshore sand bars has been prescribed by a given three-dimension equation. Data sets of different surveying intervals between the running lines perpendicular to the coastline and the sampling distance in each running line are then directly computed from the given equation. The resultant difference caused by the three interpolation algorithms has been studied by using different data sets.

2. Data Sampling and Processing

A typical bar-type terrain has been given by the following equation,

$$Z = -0.1X^{2/3} - \mathrm{SIN}(X/120) - \mathrm{SIN}(Y/300+5/3) \qquad (2\text{-}1)$$

where X: on-offshore distance from shoreline

Y: alongshore direction

Z: water depth

A 2 x 2 km terrain was taken for simulation. The resultant contours and 3-D perspective view of the given terrain can be referred to Figs. 2-1 and 2-2. The bathymetry shows two alongshore sand bars in the study region with mild undulation in the alongshore direction.

Three typical values, 50, 100 and 200m, were used for surveying interval, i.e. DY, between running lines perpendicular to the shoreline while the sampling distance, i.e. DX, in each running line is increasing from 10 to 200m with 10m increment. On the other hand, DY is increasing from 10 to 200m as DX are taken as 50, 100, and 200m, respectively, to simulate a surveying pattern with running lines parallel to the coastline. There are 120 sampling data sets directly computed from Eq.(2-1) and about three hundred sixty resultant data sets or contour maps were then obtained by three interpolation algorithms, namely, Surfer® -InvDist, Surfer® - Kriging, and Surfer® -MinCurv.

In order to study the dependence of the accuracy on the surveying interval and sampling distance, and on the interpolation algorithms, the depths of 25 randomly taken points in 25 equally distributed squares were computed from each resultant data set and the error was determined with respect to the theoretical depth given by Eq.(2-1). The average and standard deviation of these 25 depths were also computed from each data set.

Results of the computed standard deviation have been plotted with respected to DX and DY for the three interpolation algorithms and shown in Fig.2-3 to Fig.2-8.

3. Results and Discussion

The standard deviation of the resultant contours may change corresponding to the change of the sampling pattern varying surveying interval DY and sampling distance DX as well as to different interpolation algorithms.

3.1. Dependence of Standard Deviation on Sampling Distance DX

The dependencies of standard deviations on the sampling distance DX and interpolation algorithms are shown in Fig.2-3 to Fig.2-5 for three interpolation algorithms, respectively. In each figure, results of three DY values, 50, 100, and 200m, are shown while DX changes from 10 to 200m with 10m increment.

As shown in Fig.2-3 to 2-5, it is found that except in Surfe®r-InvDist the standard deviation increases as the sampling distance increases, almost linearly, as its value is greater than about 100m. The standard deviation of Surfer®-InvDist on the other hand has an increasing trend, with fluctuation, as DX increases. However, it is worth to note that in Surfer®-InvDist and Surfer®-Kriging the standard deviation may increase as DX decreases if the DX value is less than some value, e.g. 30m, in this study for the cases of DY = 100 and 200m. This may be a quite surprising and contrary result to people who believe that finer sampling distance should result in a better contour mapping.

Results also show that there is almost no distinguishable difference in standard deviation for Surfer®-Kriging and Surfer®- MinCurv for the cases of DY of 50 and 100m. This is different from that of Surfer®-InvDist in which the differences in three varying surveying interval DY are obvious. Besides, in each algorithm, the standard deviation for different DY seems to converge to the same value as DX reaches to 200m.

3.2 Dependence of Standard Deviation on Surveying Interval DY

Standard deviations also depend on the survey interval DY and the results are shown in Fig.2-6 to Fig.2-8 while applying different interpolation algorithms. Similar to those for varying DX, results of three DX values, 50, 100, and 200m, are shown in each figure while DY changes from 10 to 200m.

As shown in Fig.2-6 and 2-7 for Surfer®-InvDist and Surfer®- Kriging, respectively, it is found that as DY is less than about 50m the standard deviation increases rapidly when the surveying interval DY decreases, especially for the large sampling distance, DX = 200m. This is similar to the results of varying DX for fixed DY and may imply that in using these interpolation algorithms unexpected error may be resulted if the sampling spacing is too small as, at least, in the cases of the study. Furthermore, as shown in Figs.2-7 and 2-8, for Surfer-Kriging and Surfer-MinCurv, respectively, it is interesting to note that standard deviations slightly increase as DY decreases for the case of DX = 200m, on contrast to that of DX = 50 or 100m.

On the other hand, it is also found that the standard deviation for DX = 200m is much larger than those for DX = 50 and 100m. This may be caused by the presence of the alongshore bar which require appropriate sampling spacing to be reconstructed. Since the surveying pattern of varying DY for fixed DX can be considered as to simulate that with running lines parallel to the coastline, it is therefore to imply that the spacing between running lines should be carefully chosen to reflect the presence of the alongshore bar, and the interpolation algorithm should be used in caution when alongshore bars are present.

4. Conclusion

By comparing the results from different sampling patterns and processed by different Surfer® interpolation algorithms, it is found that the Surfer®-MinCurv is most insensitive to the change of sampling spacing than the others, Surfer®-InvDist and Surfer®- Kriging. This may be because that the study terrain is given by a continuous function which can be better interpolated by a continuous function such as used by Surfer®-MinCurv. Therefore, Surfer®-MinCurv provides best fit to the given topography with reasonable sampling spacing, such as DX and DY between 50 to 100m in the study. However, this range may change corresponding to the characteristics of the alongshore bars.

It is found that the standard deviation may increase and unexpected error may occur if the sampling space either in DX or DY is too small, e.g. about 30m in the study, when applying the Surfer® software. On the contrary, if the sampling spacing of DX or DY is greater than, e.g. about 100m, the standard deviation will increase as the spacing increases.

From the results of the study, it is shown that interpolation algorithms may be sensitive to the surveying pattern such as the sampling spacing as well as the running direction. It is therefore suggested that a newly received contouring software should be used with caution and some pre-study on the characteristics of the field topography such as the size of the alongshore bar should be done before a surveying pattern is scheduled.

5. Acknowledgements

The authors are grateful to Mr. Kong-Wu Chang, Ms. Yea-Fung Wong, and Mr. Kung-Jer Yang for their great patient on the tremendous work of data processing, figure drawing, etc..

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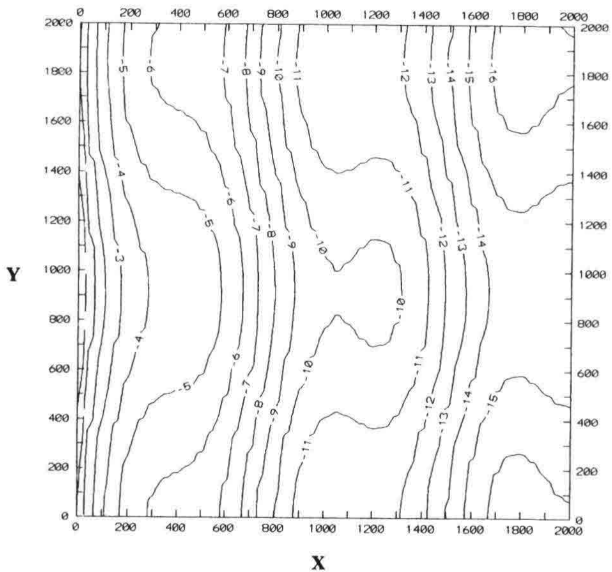

Fig 2-1 The contours of the given bathymetry

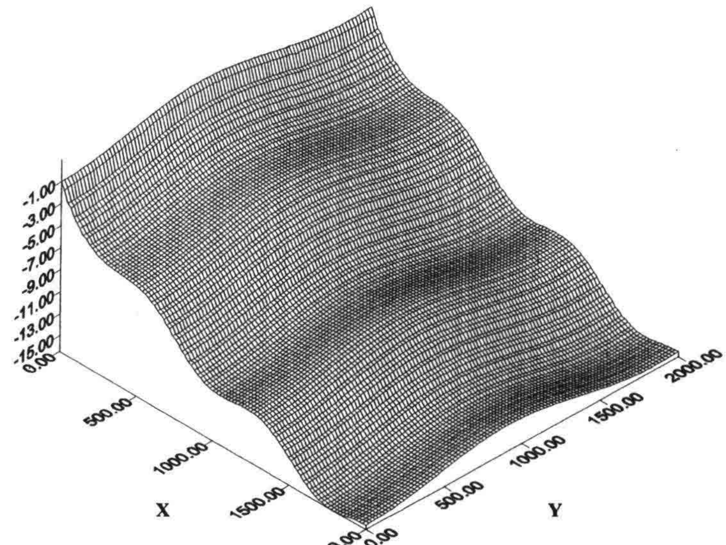

Fig 2-2 The 3-D perspective view of the given bathymetry

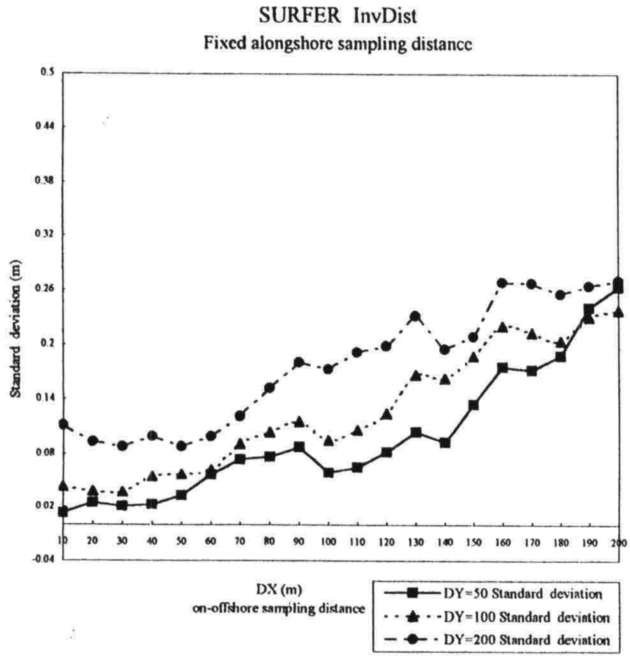

Fig 2-3 Dependence of standard deviation on on-offshore sampling distance DX , Surfer-InvDist

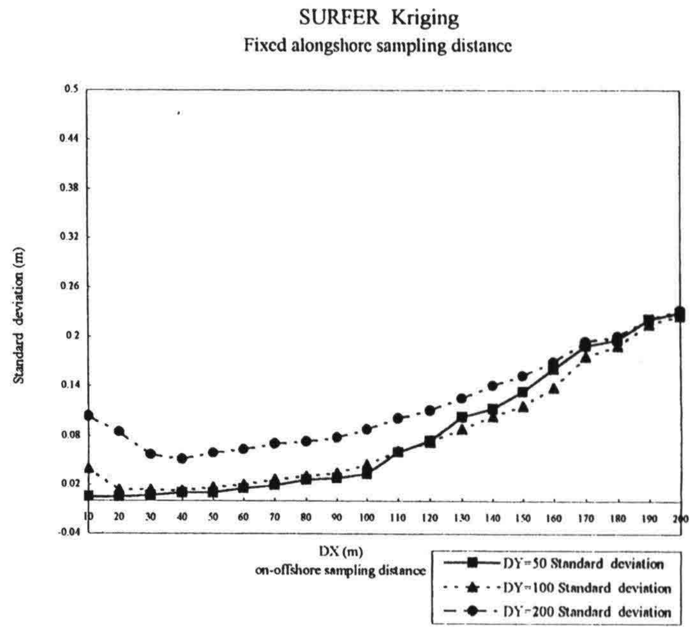

Fig 2-4 Dependence of standard deviation on on-offshore sampling distance DX , Surfer-Kriging

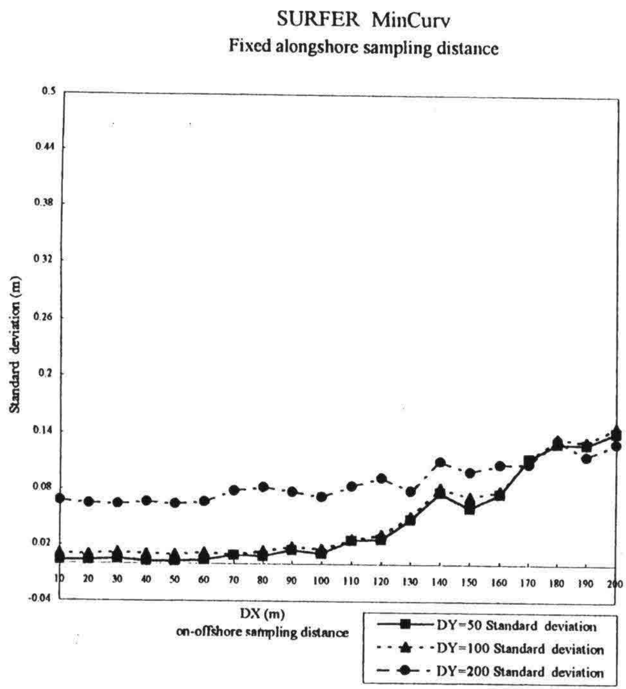

Fig 2-5 Dependence of standard deviation on on-offshore sampling distance DX , Surfer-MinCurv

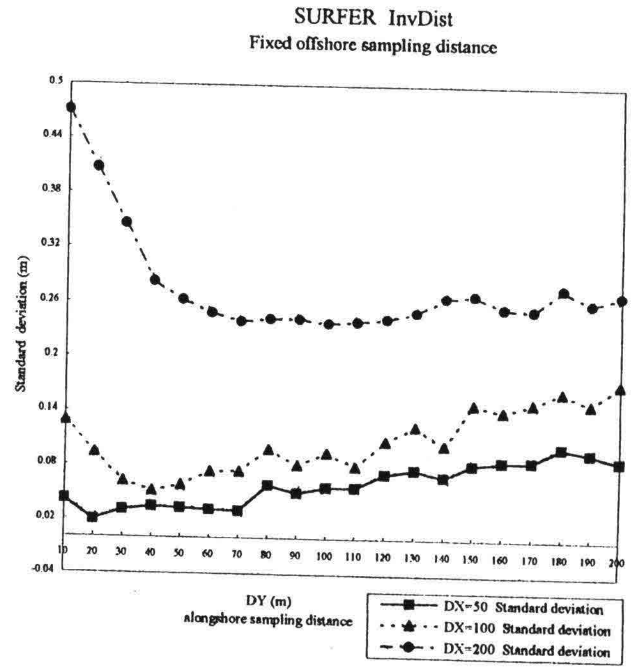

Fig 2-6 Dependence of standard deviation on alongshore sampling distance DY, Surfer-InvDist

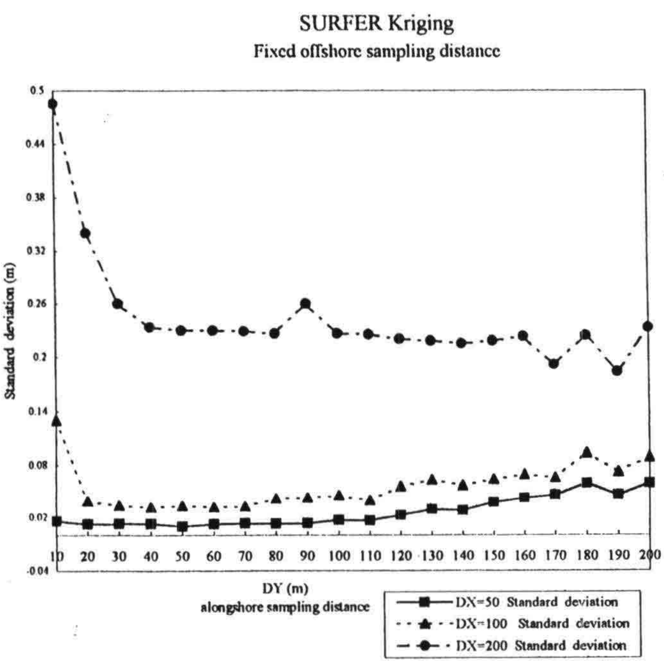

Fig 2-7 Dependence of standard deviation on alongshore sampling distance DY, Surfer-Kriging

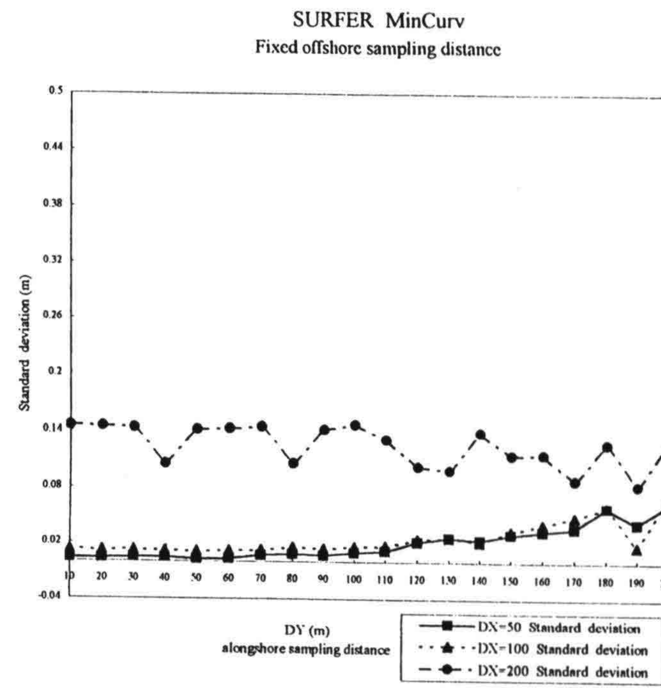

Fig 2-8 Dependence of standard deviation on alongshore sampling distance DY, Surfer-Mincurv

Recent developments in shallow water wave prediction.

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Abstract

Within the last decades considerable progress was achieved in the prediction of surface gravity waves. At many weather services numerical wave models are in operational use for sea state prediction up to 10 days for the deep global oceans and for shallow regional seas. Typical resolution are about 1.5° for the global models and 5 km to 50 km for the regional seas. The physical basics of the models will we be discussed and model applications will be presented.

The results of a hindcast study and the evaluation of the wave data set for the reconstruction of the Warnemünde Harbour entrance will be presented in detail.

1 Introduction

Ocean surface gravity wave store huge amounts of energy. This energy can be transported over large distances before it is finally released by wave breaking in the coastal zone. Waves influence the human activities everywhere in the open ocean, in coastal areas, and in lakes. In particular shipping, off-shore and coastal constructions are exposed to the wave forces. Therefore the two important applications of numerical wave modelling are:

- The wave forecast in analogy to weather prediction which is important for all operations on sea, e.g. for the routing of ships and the operations on oil rigs.

- The wave hindcast uses historical wind data to reconstruct the sea state. This is done for special events (mostly disasters) and to get the basic data for climate- and extreme value statistics e.g. for the layout of constructions.

In addition to the traditional applications described above in recent years numerical wave models are applied as part of the coupled atmosphere ocean models. For large scales like climate simulation over many year this is necessary to describe the fluxes of momentum, heat, moisture, through the sea surface correctly. For meso and small scale modelling (e.g. high resolution coastal area models) the interaction between waves and currents and waves and the sea floor are the dominant processes. These have to be taken into account to simulate correctly the wave induced current, and the wave forces acting on the sea flour resulting in erosion of sediment.

Two types of numerical models are used to simulate the wave evolution:

- The phase resolving or deterministic models are applied to compute the wave evolution for rapidly varying conditions (within distances of the order of a wave length or less). The models reconstruct the sea surface evolution in space and time with high accuracy. The advantage of these models is that they account for diffraction effects and non-linear wave interaction effects are included in these models. It is possible to include non- linear dissipation effects, but generation by wind is usually neglected. The absence of a wind forcing and the required very high space and time resolution much finer than the resolved wave length and period, respectively, does not allow an application to large areas. Examples of these models are described in Madsen and Sørensen, 1992 and Berkhoff, 1972)

- The phase averaged or spectral wave models assume that wave properties do not rapidly change within a few wave length and periods. These models describe the sea surface by the action density spectrum. The models include all linear and non-linear wave processes, but can not simulate diffraction. Because they allow a much coarser resolution in space and time, spectral models can be used over large areas and compute the sea state over very long periods.

In this paper spectral wave models will be addressed. The rapid progress in the last 20 year made it possible to apply reliable models in from global to coastal scales. In the following chapter the basics of these models is shortly outlined and in the following a few typical examples will be presented.

2 Spectral Wave Models

A detailed description of the physics and numerical implementation of the state of the art of spectral wave models is given in Komen et. al., 1994. Since Gelci et. al., 1956 wave models are based on the balance equation of action density N(t, **x**, **k**)

$$\frac{\partial N}{\partial t} + \nabla_x \bullet \dot{\mathbf{x}} N + \nabla_k \bullet \dot{\mathbf{k}} N = S \qquad (1)$$

where $\mathbf{x} = (x_1, x_2)$ and $\mathbf{k} = (k_1, k_2)$ are the location and wave number vector, respectively. The velocities, which account for the wave propagation including refraction by bottom and currents,

$$\dot{\mathbf{x}} = +\nabla_k \Omega$$
$$\dot{\mathbf{k}} = +\nabla_x \Omega \tag{2}$$

are defined by the dispersion relation

$$\Omega = 2\pi f = \sigma + \mathbf{k} \bullet \mathbf{v} \tag{3}$$
$$\sigma = \sqrt{gk \tanh(kd)} \tag{4}$$

which connects the angular frequency Ω or the frequency f with the wave number **k**, the current velocity **v** and the water depth d.

The source function S is the sum of all the different process changing the local balance of the action density.

$$S = S_{in} + S_{dis} + S_{nl} + S_{bot} \tag{5}$$

S_{in} denotes the input from the atmosphere, S_{dis} the energy loss due to dissipation (wave breaking), S_{nl} the nonlinear energy transfer due to conservative interactions, and S_{bot} the energy loss due to bottom interactions.

The dispersion relation is used to transform action densities into energy densities which are functions of frequency and direction. From these spectra sea state parameters i.e. significant wave height, mean periods and directions are deduced by integration. Information about sea and different swell systems is possible.

A number of computer programs exists to solve the basic equ. 1. They differ in the numerical integration scheme and in the form of the source functions. At GKSS Research Centre the HYPA (Günther et. al. 1979) is applied for deep water wave modelling and the HYPAS (Günther et. al. 1984) for shallow water areas. Both models are second generation models and parameterise the source functions of growing wind waves with the JONSWAP (Hasselmann et. al. 1973) or by the TMA (Bouws et. al., 1985) spectral form. This results in different numerical schemes for sea and swell, which is typical for second generation models. These models require much less computer resources than the third generation WAM model (Komen et. al.,1994, Günther et. al. 1992) which uses explicit forms for all the source functions. Therefore a sea - swell separation is not necessary in this model. At GKSS the model was successfully applied in a hindcast study within the WASA project to estimate wave height trends in the North East Atlantic over the past 40 years (Günther et. al., 1997). Fig. 1 shows the trend of the highest 10% of sig. wave heights computed from the WASA data set in the North East Atlantic. Whereas in the Northern North

Sea an increase of up to 1 m was found the wave decrease by about the same rate West of Ireland.

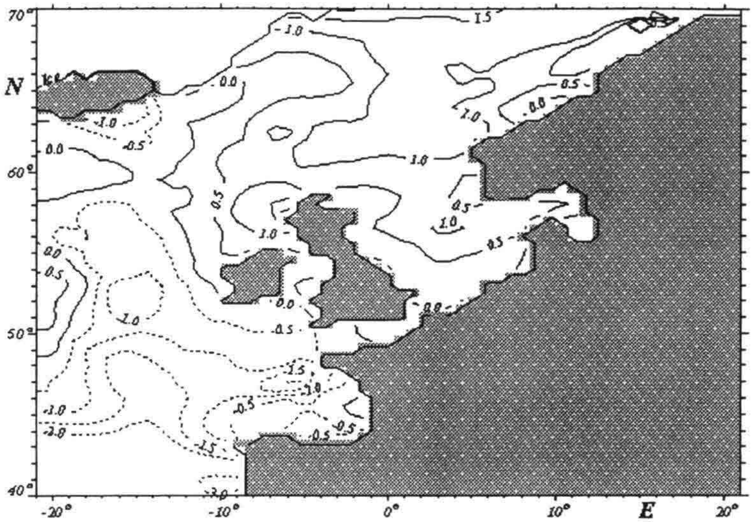

Fig. 1: Trend of the 90 percentile of sig. wave height in the North East Atlantic estimated form the WASA hindcast over the period 1955 - 1994. Contour lines are change per year in cm.

3 Examples

3.1 Wave forecast

The transport equation is numerically solved on a discrete grid. Fig 2 (from Behrens and Schrader, 1994) shows the grid configuration as used by Bundesamt für Seeschiffahrt und Hydrographie (BSH) and German Weather Service (DWD) since June 1992 for the operational wave forecast. The system includes the deep water model HYPA for the North Atlantic (resolution 150 km) as well as the shallow water model HYPAS for the North European Shelf (resolution 30 km) and the Baltic Sea (resolution 15.875 km). These models are driven by wind fields from atmospheric models of the DWD and supply daily forecasts for the North Atlantic 7 days in advance and forecasts for the North European Shelf and Baltic Sea 3 days in advance. The computed wave data are used for the tasks of the various services and for shiprouting consult as well.
This model system has been verified manifold (Behrens and Schrader, 1994).

Figure 3 shows the comparison between computed and measured sig. wave heights at Research Platform North Sea (see Fig 2). Both measured and computed values agree well. The mean deviation computed for 1 year was 15 cm and the standard deviation was less than 46 cm. Meanwhile these values, in particular the mean deviation, could be reduced by an improved resolution and a better quality of wind fields.

3.2 Wave hindcast

The wind fields necessary for driving a wave model with forecast computations are nowadays available at the Weather Services in good quality as opposed to applications with historical wind fields for the so-called hindcast, since digitized wind fields in an appropriate spatial (100 km or less) and temporal (3 - 6 h) resolution have been produced and stored only during the last few years when the regional weather models have been introduced.

The main target of the application of wave hindcast is to produce basis data for statistical investigations, as there are either no field measurements at all in the interesting sea areas or there are no measurements for a sufficient period of time which would allow a statement of any statistical significance. A minimum period for climatological statements would be at least 5 years, 20 - 40 years for extreme value analysis of the strongest storms. Therefore it is necessary that the driving wind fields are consistent in time. But this is hardly given in the routine fields of the weather services, if they are available at all, because the methods of computing wind have been improving steadily during the last years.

Subsequently we will show the typical proceeding in sea wave hindcast at the example of the project "Construction of a climatology and extreme value analysis for the Baltic Sea in the area of Warnemünde harbour" (Gayer et al. 1995). This project was performed by GKSS on behalf of Bundesanstalt für Wasserbau (BAW) in order to gain basic wave data for the allocation of buildings during the reconstruction of the harbour entrance. The results of this hindcast have been used to drive their physical model of Warnemünde Harbour.

3.2.1 Climatological statistics

For the generation of climatological statistics the waves were continuously hindcasted during the years 1988 - 1993 by the wave model HYPAS on the Baltic Sea grid (see Fig 2). The wave spectra and the integrated parameter (significant wave height, mean and peak wave period and wave direction) were stored every 3 hours at all grid points in the German area. The model was driven by wind fields from the analysis of the German Meteorological and Geographical Office. These fields were available every 6 hours on a grid with a resolution of 127 km and were interpolated bilinearly in space. In time the fields were interpolated on the model time step of 15 min by means of a procedure which takes into account the advection of cyclones. These wind fields

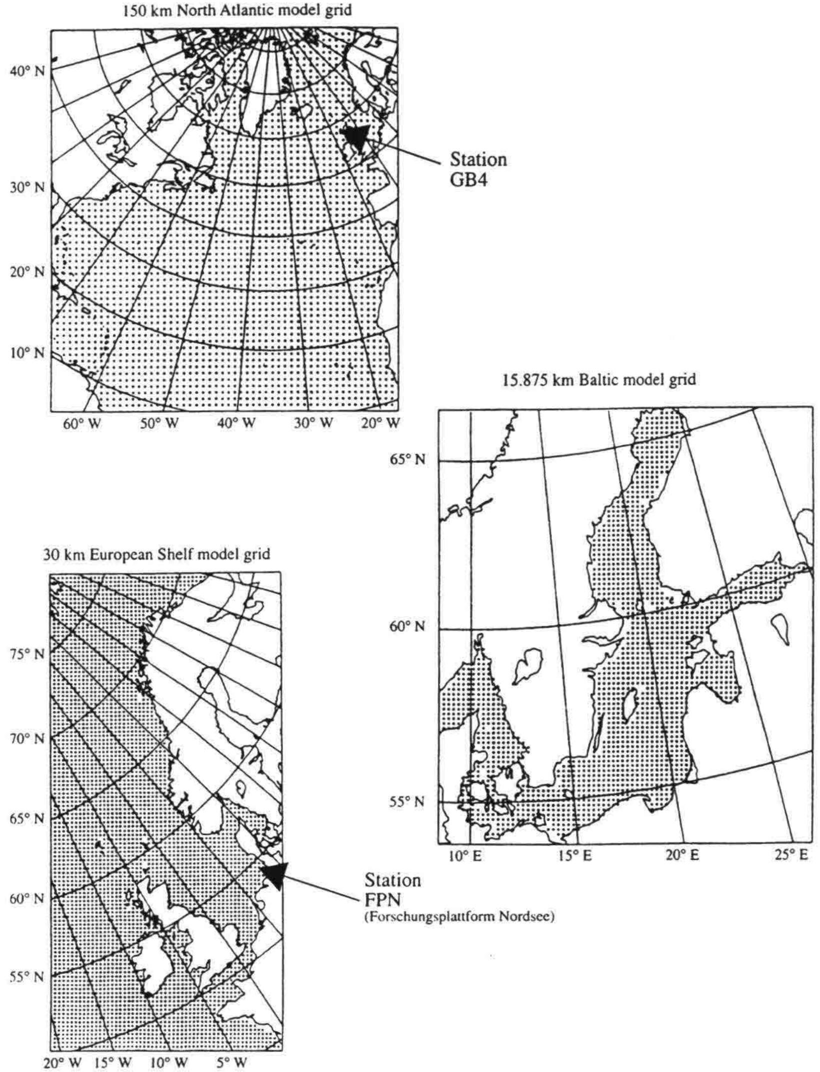

Fig 2: Grid of the BSH-DWD routine forecast system for wave computations.

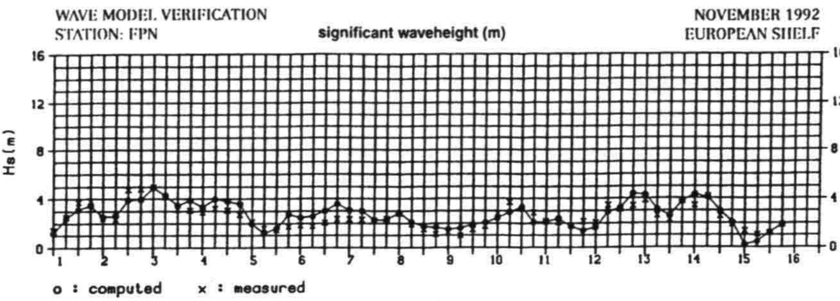

Fig 3: Time series of measured and computed sig. wave height at Research Platform North Sea from 1 - 16 November, 1992. (from Behrens and Schrader, 1994)

have also been used to force the wave model and one of its remarkable features is its good consistency since it has been drawn high attention to the demands of a good wave forecast.

For the verification of the system comparisons were made with buoy measurements at Zingst and in front of Warnemünde. Table 1 shows the statistics of the comparison between computed and measured sig. wave heights (H_s) at Warnemünde during the months February and March 1993. The highest measured wave height (y_{max}) in these 2 months is with 2.11 m 11 cm above the computed value (x_{max}). While the mean values (x_q and y_q) differ during the both months by d_q = 10 cm and -7 cm. The difference of -1 cm over the whole period can be faintly noticed. The scatter of the measurements (S_y) and computations (S_x) about their mean values is about 53 cm. The only exception are shown by the measurements in March with a scatter of 33 cm. The standard deviation between measurement and model are smaller than the scatter of the particular data sets during the whole period with S_d = 29 cm in February, 35 cm in March and 33 cm over the full period. The scatter index (Scat) of 46 %, scaling the standard deviation with the mean value of the measurements and the correlation (Cor) of 0.80 can be accepted at this range of wave heights. Even according to international standards (standard deviation (S_d) smaller than 50 cm at significant wave heights below 5 m and nearly no systematic deviation (d_q)) a good agreement between computation and measurement has been achieved at the station Warnemünde. Some according comparison statistics have been achieved for the station Zingst as well.

No.	y-axis	x-axis
1	sig. wave height	Peak period
2	sig. wave height	TM2 period
3	sig. wave height	wind speed
4	sig. wave height	wave direction
5	wind speed	wind speed
6	sig. wave height (WMO)	wind speed (WMO)
7	sig. wave height	duration
8	wind speed	duration

Tab. 3: Produced Diagrams

3.2.2 Extreme Value Analysis

The above mentioned data set is not suitable for an extreme value analysis, e.g. the estimation of the largest expected wave height 100 years ahead, as the data set contains only few storms. In general data of 20 - 30 extreme wave events are needed for this extrapolation to achieve some significant estimations. As the computation of waves and in particular the generation of the appropriate wind fields is very costly the computation was limited to selected storms which were considered to be relevant for the sea area considered.

In this project the wind measurements of the Meteorological Station Warnemünde from 1954 - 1993 were used. 402 storm events (wind speed more than 15 m/s for more than 1 hour) were identified. The greatest risk for the harbour entrance of Warnemünde are storms from northern directions, so that only the 39 strongest storms out of these directions were taken into account.

There were no digitized wind fields available, so the analysed pressure charts of Seewetteramt Hamburg were digitized and the wind fields were computed with a boundary layer model. Fig 5 shows a digitized pressure field and Fig 6 the computed wind field. On average 17 charts were treated for each storm. This is according to a computation period of 4 days/storm and thus contains the growth and decay phase for the waves. The waves were computed from these wind fields by the HYPAS model in the same way as for the 5 year hindcast. However, the output was made hourly to determine the maximum wave height for each storm.

In order to record the extreme sig. wave heights the 20 largest events were used for the analysis. These are shown in Fig 7 at grid point Warnemünde Harbour together with the peak period, the direction sector where the waves are coming from and the storm number. The figure shows that sig. wave heights up to 3.6 m were reached, during the storm 180 (December 1957)

sig. wave height (m)	peak period (s) 2,0	2,4	2,8	3,2	3,6	4,0	4,4	4,8	5,2	5,6	6,0	6,4	6,8	7,2	7,6	8,0	8,4	8,8	9,2	9,6	10,0	≥	Σ	sig. wave height (m)
≥																								≥
5,0																								5,0
4,8																								4,8
4,6																								4,6
4,4																								4,4
4,2																								4,2
4,0																			1				1	4,0
3,8																								3,8
3,6																	6	1	1				8	3,6
3,4																1	5		1				7	3,4
3,2															1	5	10	1	1	1			19	3,2
3,0															37	13	9	5					64	3,0
2,8														7	35	5	7						54	2,8
2,6													41	108	64	23	17						253	2,6
2,4											1	5	192	52	28	4							282	2,4
2,2										2	7	194	157	46	6								412	2,2
2,0									1	4	19	170	50	11									255	2,0
1,8								1	20	304	357	285	58	1									1026	1,8
1,6								26	210	465	75	13	1										790	1,6
1,4							31	176	654	65	3												929	1,4
1,2						60	283	561	134	2													1040	1,2
1,0					76	627	565	103	9	2													1382	1,0
0,8				535	1023	476	105	25	3	1					1								2169	0,8
0,6				811	330	118	25	26	2			2											1314	0,6
0,4				1268	49	63	76	70	18	7	4	9		3		1							1568	0,4
0,2	1195			1067	180	267	89	76	26	8	8	12	7	4	5	2	1	1					2948	0,2
Σ	1195			3681	1658	1611	1174	1064	1077	860	474	690	506	232	177	54	55	8	4	1			14521	Σ
	2,0	2,4	2,8	3,2	3,6	4,0	4,4	4,8	5,2	5,6	6,0	6,4	6,8	7,2	7,6	8,0	8,4	8,8	9,2	9,6	10,0	≥	Σ	
	peak period (s)																							

Tab.2: Distribution of the significant wave height versus the peak period. The number of events is shown at the grid point Warnemünde Harbour, which occurred during the computed 5 years .

Month 1993	Number	Model x_{max} (m)	x_q (m)	s_x (m)	Buoy y_{max} (m)	y_q (m)	s_y (m)	Buoy - Model d_{max} (m)	d_q (m)	s_d (m)	Cor	Scat %
2	99	2,00	0,69	0,53	2,11	0,79	0,52	0,79	0,10	0,29	0,84	42
3	176	1,90	0,73	0,54	1,71	0,66	0,33	0,79	-0,07	0,35	0,77	49
Total	275	2,00	0,72	0,54	2,11	0,71	0,40	0,79	-0,01	0,33	0,80	46

Tab. 1: Statistics of the comparison between computed and measured sig. wave heights at Warnemünde during the February and March 1993.

The generated time series, consistent of 14521 values were used at selected grid points to generate climatological statistics. Table 2 displays the distribution of the sig. wave height (H_s) and the peak period at the grid point Warnemünde Harbour. The highest wave height was 3.8 m - 4.0 m and had a period between 8.8 s and 9.2 s. The longest period was between 9.2 s and 9.6 s and was in accordance with a wave height of 3.0 m - 3.2 m. Both these events occurred exactly one time. In 3681 events periods between 2.8 s and 3.2 s arose most frequently. These were in accordance with wave heights smaller than 0.8 m whereas the largest number was smaller than 0.4 m. Periods smaller than 2.8 s and wave heights below 0.1 m were not resolved by the model. This situation could be noticed 1195 times and was marked in the smallest class of wave heights and periods.

The mutual distribution shows two groups of entries. The first group is marked by an increase of wave heights in longer periods. This is a typical characteristic for waves which are influenced by the wind. The second group shows that at small wave heights long periods can arise as well. This group matches with swell events.

The appropriate diagrams were not only produced for the whole period of 5 years but also separately for the four seasons. Along with the discussed climatological diagram wave height - wave period the distributions shown in table 3 were evaluated as well.

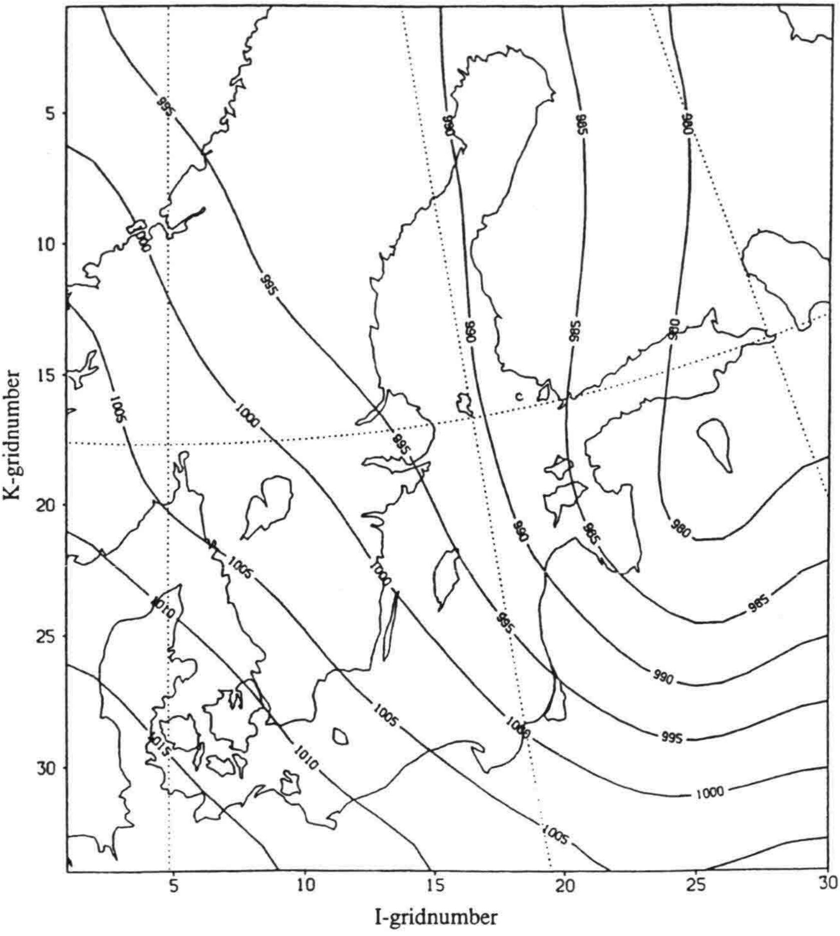

Fig. 5: Isobars of the digitized pressure field of 23. November 1973 at 12:00 which was interpolated to the wave model area.

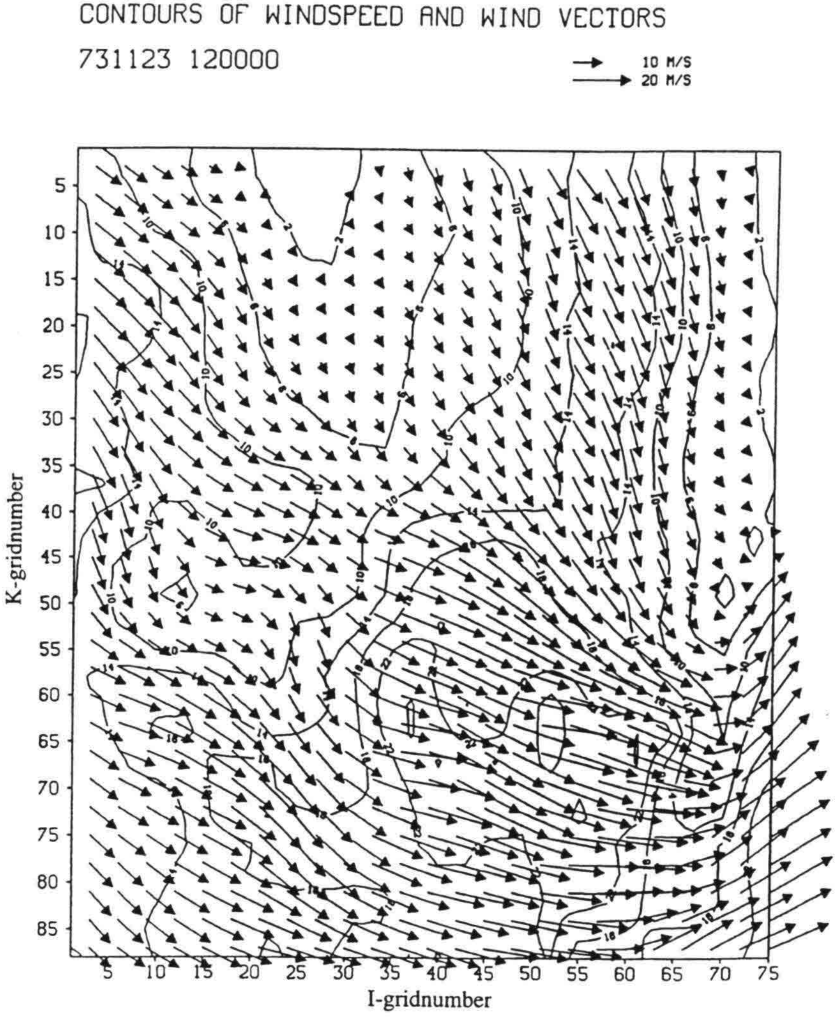

Fig. 6: Isolines of wind speed and wind vectors of the wind field on 23 November 1973 at 12:00 computed from the pressure field in Fig. 5. For a better survey the wind vectors were shown only for every 3rd model grid point.

from direction NNO with a peak period of 10.1 s and during storm 3 (January 1968) with a peak period of 9.0 s from North.

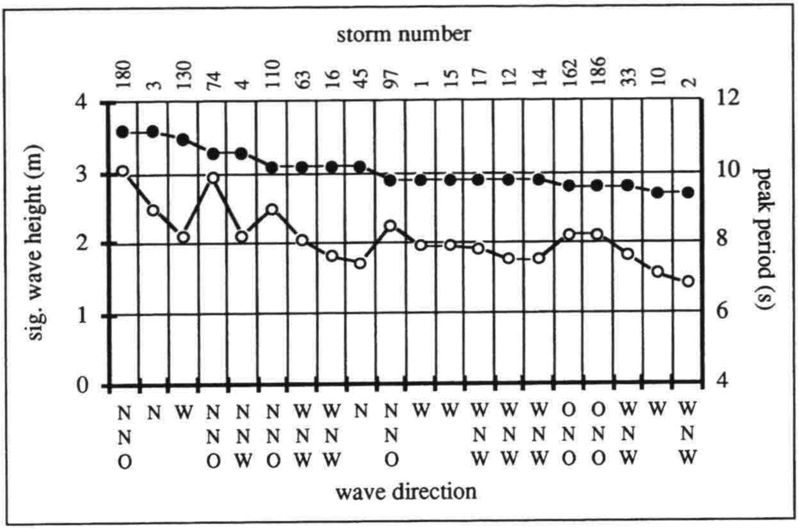

Fig. 7: Maximum significant storm wave heights (•) at the grid point Warnemünde Harbour and the adherent peak period (o) and wave direction. The 20 largest events in descending order are shown.

Out of these data the expected value of the 25, 50 and 100 years return value for the sig. wave height was extrapolated from various extreme value distributions. Table 4 summarizes the results received for the grid point Warnemünde Harbour. It contains the 25, 50 and 100 year return wave heights and esti-

Name	25 year return				50 year return				100 year return			
	Hs (m)	Hl (m)	Hr (m)	Δ (m)	Hs (m)	Hl (m)	Hr (m)	Δ (m)	Hs (m)	Hl (m)	Hr (m)	Δ (m)
FRECHET	3,58	3,44	3,72	0,28	3,78	3,62	3,94	0,32	3,99	3,82	4,18	0,36
GUMBEL	3,56	3,43	3,69	0,26	3,73	3,60	3,87	0,27	3,90	3,76	4,05	0,29
LOG-NORMAL	3,49	3,31	3,68	0,36	3,60	3,41	3,80	0,38	3,70	3,51	3,91	0,41
NORMAL	3,48	3,30	3,66	0,35	3,58	3,40	3,76	0,36	3,66	3,48	3,85	0,37
WEIBULL	3,39	3,15	3,65	0,49	3,45	3,20	3,71	0,51	3,49	3,24	3,77	0,52

Tab. 4: The extrapolated 25, 50 and 100 year return wave heights (Hs) for various extreme value distributions. Hl is the lowest and Hr the upper value of the error margin. The width of the interval is Δ.

mates of the error margin. The Weibull-distribution shows the greatest error margin (0.52 m for the 100-year wave of 3.49 m). The Gumbel-distribution gives the smallest error limits with 0.29 m for a 100-year wave of 3.90 m. The largest estimated value (3.99 m) for the 100-year wave results from the Frechet-distribution with an error interval with of 0.36 m.

4 Summary

We have shown two examples for the application of numerical wave models. First of all the wave forecast was introduced which is used for shiprouting consult and for the planning of any marine activity. Furthermore the wave hindcast and its evaluation for a wave statistic was discussed. Since there are hardly any sufficient field measurements available, this method has turned into a standard procedure for several applications, e.g. for oil platforms, coastal protection and constructions.

During the last few years the numerical wave modelling has become an important tool of increasing importance as a part of coupled modelling in coastal zones. Especially the modelling of erosion and deposition of sediment is only possible with detailed wave computations.

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Forecasting of Typhoon Waves for a Harbor at Eastern Coast of Taiwan

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Abstract

The typhoon waves coming with storm surges play a major role on damages of coastal structures and harbor constructions in Taiwan area, especially at eastern coasts of Taiwan. A new harbor will be constructed at eastern coast of Taiwan. Four sites were chosen for further evaluation. This study was to develop the theoretical methodology and numerical techniques of forecasting the significant wave heights and periods caused by typhoons. The waves generated by typhoons were considered in moving fetches and the numerical shallow water wave calculation model for typhoon wave prediction was also adopted in this paper.

1 Introduction

Typhoons always threaten Taiwan during the summer and autumn seasons every year and some of them often caused heavy damages on coastal structures and harbor constructions, especially at eastern coast of Taiwan. Typhoon waves play a major role on wave climates during typhoon seasons. Also, the impact of storm surges are the serious disaster to the coastal area of Taiwan. Many researches have shown that the serious impacts are due to large waves associated with swells generated by typhoons. At the same time there was an extraordinary sea level rise lasted for hours to induce waves to reach a higher level of coast lines. This inundation caused by storm surges always threatened the safety of lives, properties and infrastructures at coastal region of Taiwan.

A new harbor will be constructed at eastern coast of Taiwan. Four sites were chosen for further evaluation, but only the influences of typhoon waves were presented in this paper for the sake of limitation of pages. This study was to develop the theoretical methodology and numerical techniques of forecasting significant height and periods of waves caused by typhoons. The waves generated by typhoons were considered in moving fetches and the numerical shallow water wave calculation model for typhoon wave prediction was adopted in this paper. In case that the waves become swell, the criteria of swell estimation equations are also applied.

2 Determination of pre-required parameters

To calculate typhoon waves, there are some parameters should be determined in advance to be applied in our forecasting model. One of them is to select typhoon velocities and directions. It is shown that in the area of Taiwan the maximum and minimum moving velocities of typhoons are 40 Km/hr and 4 Km/hr, respectively, according to data recorded by the Central Weather Bureau. The most likely value and the average value are 18~20 Km/hr and 20 Km/hr, respectively, so in the following calculation the moving velocity of typhoons was assumed to be a constant of 20 Km/hr.

The past data shown that there are 362 typhoons have ever threatened Taiwan in the period of 1897~1995. Their paths could be classified into seven types as shown in figure 1, in which types I, II, III and IV will induce the typhoon waves of more important influence on the eastern coast of Taiwan. Therefore, we simulated the typhoon paths based on these four types as shown in figure 2 to be the design typhoons.

To simplify the calculation and to satisfy the practical application, the wave directions were classified to 16 divisions (22.5° for each). However, if the boundary conditions were considered, the possible wave directions will be limited to some ranges for each reserve site of harbors, for example, they may come from the directions of NNE, NE, ENE, E, ESE, SE, SSE and S for site A in different typhoons.

Another very important factor is wind velocity along the fetch line. Instead of computing the wind velocity of each lattice point of space-time wind diagram, a moving fetch method was applied to determine the wind velocity on the fetch line in case the typhoon record was available (Tang 1981). Before calculating the wind velocity we have to determine the pressure coefficient, ΔP, of the following formula

$$P = P_c + \Delta P \exp[-R_0 / R] \qquad (1)$$

in which P is pressure at the point at a distance of R from the typhoon center.

P_c is center pressure of the typhoon.

ΔP is pressure coefficient

R_0 is distance from typhoon center to the circle where maximum wind velocity occurs.

We have collected 64 records of typhoons happened in the area where we are concerned. To determine the biggest pressure difference at the center of typhoons for a given return period, we assumed that the pressure difference, dP, has a Log-Pearson Type III Distribution. On this basis the pressure coefficient, ΔP, for a return period of 100 years could be determined in the following steps:

1. The pressure differences, dP, were sorted as variables y_i (i=1, 2, 3, -- ,N), and to be represented as $x_i = \log y_i$.
2. Calculate the mean, variance and skewness of variables x_i ,

Mean $$\bar{x} = \frac{1}{N} \Sigma \ \log y_i \qquad (2)$$

Variance $$s = [\Sigma (x - \bar{x})^2 / N - 1]^{1/2} \qquad (3)$$

Skewness $$C_s = N \Sigma (x - \bar{x})^3 / [(N-1)(N-2)s^3] \qquad (4)$$

3. Now the variables x_i of the probability p that in a given time period could be expressed as

$$x = \log y = \bar{x} + s \cdot K \tag{5}$$

in which

$$K = t + (t^2 - 1)(\frac{C_s}{6}) + \frac{1}{3}(t^3 - 6t)(\frac{C_s}{6})^2 - (t^2 - 1)(\frac{C_s}{6})^3 + t(\frac{C_s}{6})^4 + \frac{1}{3}(\frac{C_s}{6})^5 \tag{6}$$

$$t \approx \begin{cases} W - \dfrac{c_0 + c_1 W + c_2 W^2}{1 + d_1 W + d_2 W^2 + d_3 W^3} & 0 < p \le 0.5 \\ -(W - \dfrac{c_0 + c_1 W + c_2 W^2}{1 + d_1 W + d_2 W^2 + d_3 W^3}) & p > 0.5 \end{cases} \tag{7}$$

$$W = \begin{cases} \sqrt{\ln(\dfrac{1}{p^2})} & 0 < p \le 0.5 \\ \sqrt{\ln[\dfrac{1}{(1-p)^2}]} & p > 0.5 \end{cases} \tag{8}$$

and $c_0 = 2.51557, \quad c_1 = 0.802853, \quad c_2 = 0.010328$

$d_1 = 2.51557, \quad d_2 = 0.802853, \quad d_3 = 0.010328$

If the record is not long enough, the skewness should be modified as

$$C_s' = C_s(1 + 8.5 / N) \tag{9}$$

Substituting the 64 records of typhoon into above procedures, we obtained the pressure coefficient ΔP =164.55mb for a return period of 100 years.

3 Typhoon wave forecasting methods

To evaluate the waves generated by typhoons, Wilson (1955, 1961, 1962) proposed graphical and numerical methods for waves in developing stage, however they are not suitable while the waves become swell and/or enter shallow water areas. For wind waves in shallow water of constant depth, Ijima and Tang (1966) presumed the following relationships, according to the researches of Bretschneider (1958) and Thijsse (1952),

$$\frac{gH}{U^2} = 0.26 \tanh[0.578(\frac{gD}{U^2})^{3/4}] \tanh\left\{\frac{0.01(\frac{gF}{U^2})^{1/2}}{\tanh[0.578(\frac{gD}{U^2})^{3/4}}\right\} \tag{10}$$

$$\frac{gT}{2\pi U} = 1.40 \tanh[0.520(\frac{gD}{U^2})^{3/8}] \tanh\left\{\frac{0.0436(\frac{gF}{U^2})^{1/3}}{\tanh[0.520(\frac{gD}{U^2})^{3/8}]}\right\} \tag{11}$$

in which H and T are significant wave height and period of waves generated by a wind velocity U over shallow water area of a constant depth, D; F is fetch length and g is gravitational acceleration.

Both of the equations are approaching Wilson's equations while $D \to \infty$,

$$\frac{gH}{U^2} = 0.26 \tanh[0.01(\frac{gF}{U^2})^{1/2}] \tag{12}$$

$$\frac{gT}{2\pi U} = 1.40\tanh[0.0436(\frac{gF}{U^2})^{1/3}] \quad (13)$$

i.e. $G/U = gT/4\pi U = 0.7\tanh[0.0436(gF/U^2)^{1/3}]$ (14)

where $0.1 < gF/U^2 < 10^6$.

Waves in shallow water will then be evaluated with stepwise method of numerical integration. First, the Wilson's formulas will be adopted to calculate waves at initial points of the fetch. If the wave height, H_n, and group velocity, G_n, at point n are known, then the wave features at point n+1 which located at a distance of ΔX leeward on the fetch line could be determined by formulas

$$H_{n+1} = H_n + \frac{dH_n}{dX}\Delta X \quad (15)$$

$$G_{n+1} = G_n + \frac{G_n}{dX}\Delta X \quad (16)$$

in which

$$\frac{dH_n}{dX} = \frac{3.846\times10^{-4}(0.26+gH_n/U_n^2)(0.26-gH_n/U_n^2)}{\ln(0.26+gH_n/U_n^2)-\ln(0.26-gH_n/U_n^2)} \quad (17)$$

$$\frac{dG_n}{dX} = \frac{1.578\times10^{-4}g(0.7+G_n/U_n)(0.7-G_n/U_n)}{U_n[\ln(0.7+G_n/U_n)-\ln(0.7-G_n/U_n)]^2} \quad (18)$$

both are for waves in deep water. If water depths becomes shallow, Eq. (10) could be differentiated to have

$$\frac{dH_n}{dX} = \frac{3.846\times10^{-4}(0.26\alpha+gH_n/U_n^2)(0.26\alpha-gH_n/U_n^2)}{\alpha^3[\ln(0.26\alpha+gH_n/U_n^2)-\ln(0.26-gH_n/U_n^2)]} \quad (19)$$

in which

$$\alpha = \tanh\ [0.578(gD_n/U_n^2)^{3/4}] \quad (20)$$

However, it is difficult to estimate group velocity in shallow water areas, because it is a function of local wave length L and depth D instead of being a unique function of wave period T in a deep water area. It may be solved by adopting the ratio between group velocity G and wind velocity U in shallow water,

$$\frac{G}{U} = \frac{1}{2}[1+\frac{2KD}{\sinh\ 2KD}]\frac{gT}{2\pi U}\ \tanh(KD) \quad (21)$$

in which K=2π/L and L is the wave length at a water depth D. Let

$$S = \frac{2\pi D}{L_0}\ ,\quad Y = \frac{2\pi D}{L} \quad (22)$$

where L_0 is the wave length in deep water, then

$$S = \frac{2\pi D}{L_0} = \frac{4\pi^2 D}{gT^2} = \frac{gD}{U^2}/(\frac{gT}{2\pi U})^2 \quad (23)$$

$$\frac{gT}{2\pi U} = \left(\frac{gD}{U^2}/S\right)^{1/2} \quad (24)$$

Finally, we have

$$S = Y\tanh Y \quad (25)$$

Substitution of Eqs. (22), (23) and (24) into Eq. (21) gives the following relationship

$$Z = \frac{G}{U}\frac{1}{(gD/U^2)^{1/2}} = \frac{G}{\sqrt{gD}} = \frac{S - S^2 + Y^2}{2YS^{1/2}} \tag{26}$$

The right side of this equation is a function of S only, and the left side of this equation could be simplified to be $G/\sqrt{gD}$ corresponding to the ratio between group velocity and phase velocity. In the case of deep water wave $D/L \geqq 1/2$, so we have $S \geqq \pi$, $Y=S$ and $Z=1/2\sqrt{S} \leq 0.2821$. If the water depth is very shallow, we obtain

$$\tanh(Y) \approx Y \quad S \approx Y^2 \quad \text{and} \quad Z = 1 - S/2 \approx 1 \tag{27}$$

The variable Z could now be called non-dimensional group velocity.

In the region of $0<S<\pi$, namely, the state of shallow water waves, relationships between Z and S can be approximately represented by following polynomials

$$Z = 1 - a_1 S - a_2 S^2 \cdots\cdots a_6 S^6 \tag{28}$$

in which a_1=0.4536 , a_2=0.0931 , a_3=0.2745

a_4=0.17033 , a_5=−0.0476 , a_6=0.005067

or

$$S = b_1(1-Z) + b_2(1-Z)^2 + \cdots\cdots + b_7(1-Z)^7 \tag{29}$$

in which $b_1 = 2.464857$, $b_2 = -7.3530\dot{5}$, $b_3 = 52.7458\dot{3}$

$b_4 = -162.\dot{2}$, $b_5 = 275.8\dot{3}$, $b_6 = -247.\dot{2}$, $b_7 = 101.19476$

If G_n and D_n are known, the value of Z_n could then be calculated by Eq. (26) and S_n by Eq. (29). The values of S_{n+1} at point n+1 become

$$S_{n+1} = S_n + (dS_n / dX)\Delta X \tag{30}$$

Substitution of Eq. (11) into Eq. (23) yields

$$S = \left(\frac{gD}{U^2}\right)\left\{1.4\tanh[0.52(\frac{gD}{U^2})^{3/8}]\tanh\left[\frac{0.0436(\frac{gF}{U^2})^{1/3}}{\tanh\left[0.52(\frac{gD}{U^2})^{3/8}\right]}\right]\right\}^{-2} \tag{31}$$

Hence

$$\frac{dS_n}{dX} = \frac{-1.578\times10^{-4} gS_n(1.4\mu+\upsilon)(1.4\mu\upsilon)}{U_n^2 \upsilon\mu^4[\ln(1.4\mu+v) - \ln(1.4\mu-\upsilon)]^2} \tag{32}$$

in which , $\mu = \tanh[0.52(\frac{gD_n}{U_n^2})^{3/8}]$ and $\upsilon = \sqrt{\frac{1}{S_n}(\frac{gD_n}{U_n^2})}$

The values Z at point n+1 could then be calculated with Eq. (28). Finally, the wave period and group velocity at point n+1 may be derived directly from Eqs. (24) and (26). That is,

$$T_{n+1} = 2\pi\sqrt{\frac{D_n}{gS_{n+1}}} \tag{33}$$

$$G_{n+1} = Z_{n+1}\sqrt{gD_n} \tag{34}$$

On condition that the wave height H and group velocity G at point n can not grow any more under the circumstances of U_n and D_n, i.e., the waves have already been larger than what the wind U_n can generate in shallow water areas of depth D_n, then the criteria of swell calculation are obliged to be applied. Tang (1981) interpreted Bretschneider's graphs for estimating swell to the following equations,

$$\frac{H_d}{H_F} = \{\cosh[0.66(\frac{F}{H_f})^{0.06}(\frac{A}{F})^{0.25}\tanh\{3.0(\frac{A}{F})^{0.3}\}]\}^{-1} \tag{35}$$

$$\frac{T_d}{T_F} = \{\cosh[1.74(\frac{2\pi F}{gT_F^2})^{-0.05}(\frac{A}{F})^{0.2}\tanh\{1.02(\frac{2\pi F}{gT_F^2})^{-0.04}(\frac{A}{F})^{0.32}\}]\}^{-1} \tag{36}$$

in which H_d (m), T_d (sec) : wave height and period after decay

H_F (m), T_F (sec) : wave height and period at the end of the fetch, i.e., begin to decay

A (m) : length of wave attenuation, i.e., decay distance

The numerical results of design waves at water depths of 20m for the chosen harbor sites in four design typhoons are presented in Table 1, in which the routes of the design typhoons were assumed to be straight lines. It is seen that most of the maximum significant heights coincide the directions with paths of typhoons. Site A may happen the smallest wave height in a design typhoon of model 3, but in other typhoon cases it always happens the highest significant height. Site A is obviously not a good choice. In consideration of credits we have in table 1, site D seems to be a better choice of smaller wave heights in most of design cases. However, if we consider that the probability of typhoon model 3 is much higher than others, referred to type III in figure 1, then the influence of the highest wave height at site D in a design typhoon of model 3 should take more evaluations.

4 Conclusions

In this study we have simulated four design typhoons to forecast the significant wave heights and periods for four reserve sites of harbors at eastern coast of Taiwan. The waves generated by typhoons were considered in moving fetches and the numerical shallow water wave calculation model for typhoon wave prediction was also adopted. However, the routes of these design typhoons were assumed to be straight lines, so this deficiency should then be remedied in consideration of varying routes and also varying central pressure of real typhoons. The numerical results we have obtained are based on some assumptions, so if the conditions change the final results may be different. However, the purpose of this paper is to express the schema and methodology on forecasting typhoon waves, so it did not go further discussion. There are even some more mechanism should be considered on calculating typhoon waves or on choosing a suitable site, for example, the impact of storm surges is a serious problem to the coast of Taiwan, but we have no more spaces to discuss it.

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Table 1 The maximum significant heights and corresponding wave periods anddirections for each site in design typhoons

models / sites	1				2				3				4				Credit
	H (m)	T (sec)	Dir	Credit	H (m)	T (sec)	Dir	Credit	H (m)	T (sec)	Dir	Credit	H (m)	T (sec)	Dir	Credit	
A	18.3	19.5	ESE	1	16.5	19.6	ESE	2	12.0	20.5	ENE	4	17.6	15.9	SSE	1	8
B	13.3	17.7	ESE	3	16.0	18.7	ESE	3	13.5	19.9	ENE	3	17.2	19.9	S	2	11
C	14.2	17.4	ESE	2	16.8	16.8	ESE	1	13.8	19.7	ENE	2	15.5	15.5	SE	4	9
D	12.8	17.5	SSE	4	13.5	16.2	ESE	4	16.2	19.5	ENE	1	16.3	17.7	SSE	3	12

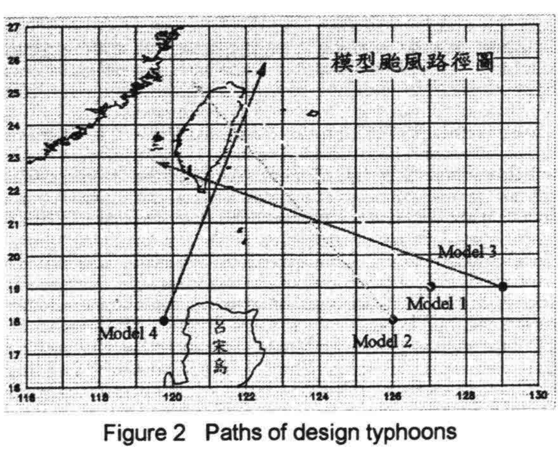

Figure 2 Paths of design typhoons

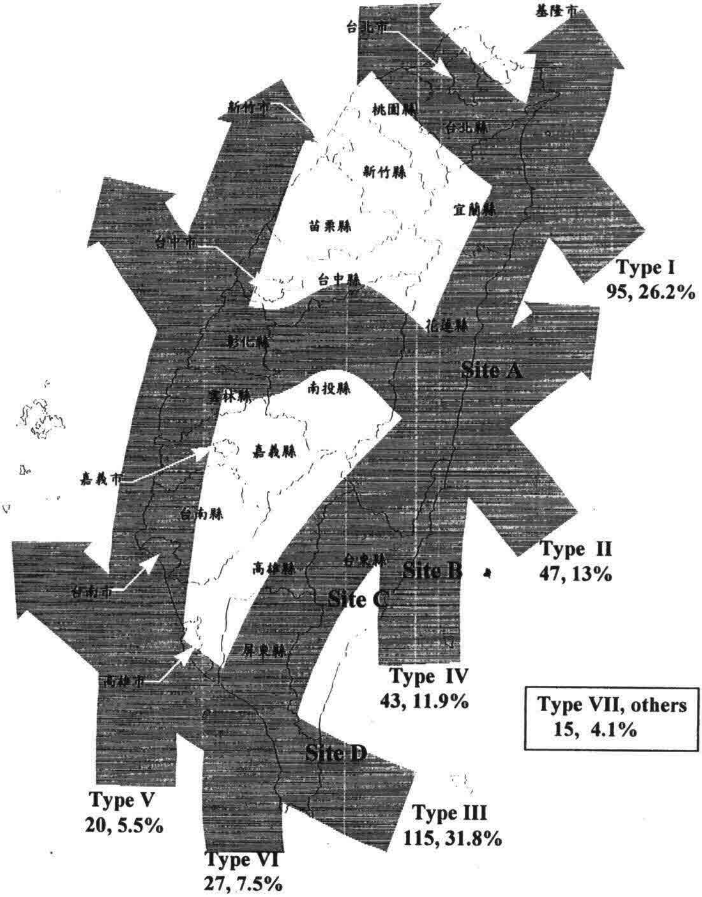

Figure 1 Classification of typhoon routes threaten Taiwan between 1897 and 1995

Topic III

Simulation Methods

Sandy Beaches, Currents and Sediment Transport

Chairman: Volker Barthel

Aspects Of Modelling Sediment Transport In Instationary Flow

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Abstract

The problem of instationary sediment transport or sediment transport under instationary flow conditions has different physical and numerical aspects. In this paper, some of the relevant aspects are discussed.

Sediments are moved by the flow of water. This may occur in coastal regions as well as in inland rivers. If the flow-velocities (and the related turbulent velocities) are high in relation to the settling velocity of the grains, sediment may be lifted up from the bed and then be transported flying. This is called suspended sediment transport. If no lift-up is present, sediment still may be moved in the vicinity of the bed. This is called bed-load-transport. The transport-mechanism is dependent on a number of parameters as there are

d = grainsize

g = coefficient of gravity

h = water depth

J_b=bed slope

v = flow velocity

v*= shear velocity = $(t/r)^{1/2}$

r' = relative density

r = density of the fluid

r_S= density of the sediment

t = bed shear stress

n = viscosity of the fluid

Additonally further effects may be of importance as for example the grain size distribution or biogene films.

Sediment transport itself may be instationary although the flow is stationary. This occurs for example when a coarse river bed is armouring. Then the grain size distribution and the absolute average grain size changes with time and size distribution and the absolute average grain size changes with time and therefore the process of transport is time dependent - instationary. On the other hand sediment transport becomes instationary when it is driven by instationary flow. Instationarity of the flow has different effects on sediment transport depending on the scale of instationarity and depending on the grain sizes. The scales may be classified as follows

- Turbulence
- Oscillatory flow (waves)
- Tides
- Flood waves

The main effect of turbulence on sediment transport is the diffusion of the water and therewith the suspension of fine sediments. Under wave motion the effects are widely similar. Sediments, especially the finer fractions, are suspended and may be transported over longer distances by weak flows, which itself would not produce sediment transport. Also without a superimposed flow a net sediment transport can be produced by wave motion. The net transport may be directed in the direction of wave travelling or in opposite direction. This depends on the wave form. In case of wave driven sediment transport absolute velocities as well as accelerations and decelerations are of importance. Special transport formulae for wave driven sediment transport are given in the literature.

In the following some aspects on sediment transport problems and methods for their solution related to tidal flows (and flood waves) are discussed.

For the calculation of sediment transport it has to be known, wat happens very close to the bed. But there is still a lack of knowlegde about the effects of the moving grain layer itself as there is its velocity and its thickness and the mechanism of lift up into suspension. Just with fixed bed our knowledge on the flow very close to a wall or a bed is rather poor.

On the other hand, the results of sediment transport, erosion and accretion produce different and often rather expensive problems. So engineers need methods to predict sediment transport altough the problem is not fully understood.

There are two ways to solve instationary sediment transport problems.

1 Use of physical models

One solution is the use of hydraulic models with movable bed. In this case lacks of physical knowledge are no longer a severe matter of evidence. Just model laws are needed and then predictions can be made from the behavior of the model. The water and the sediment in the model „know“ theirselves, how to move. To fulfill the model rules for similarity, the Grain-Reynolds-Number $Re^* = v^* \cdot d/n$ and the Grain-Froude-Number $Fr^* = v^{*2}/(r' \cdot g \cdot d)$ must be equal in model and nature. This is possible only in the case of hydraulically full rough flow in the nature as well as in the model. This ideal case only occurs, if the bed material in the nature is rather coarse and the model is not too small. But, in coastal regions, sediments are often fine and flow is hydraulically smooth in the model or even in nature. Then it is no longer allowed to reduce the model sediments in the same relation as the water depth. A model law can now only be developed with restictions, for example allowing the relation between water depth and roughness elements, the relative roughness h/d, not beeing equal in nature and model. The main result is, that the grains in the model now have to be larger than in nature but must have less density. Clearly, the relations of bed-load and suspended load are effected and the models results become more unsharp.

2 Use of numerical models

The second way to investigate complex sediment transport problems is the use of numerical models. In opposition to the use of physical models now the process of transport has to be formulated in mathematical equations. These equations have to be implemented into a numerical flow model. If not only the transport itself is calculated but additionally the bed changes are computed, such a model is called a Morphodynamic (Numerical) Model.

The analytical base for transport calculations are

; transport formulae and

; entrainment formulae

3 Transport formulae

There may be some 30 transport formulae which all are more or less empirical. The formulae are all fitted to measured data (from experiments in nature and in laboratories). If the properties of flow and sediment in a special study are different to that used to calibrate the transport formula, accuracy may become weak. But, what is good or weak accuracy ? Some studies where carried out comparing different formulae versus measured data. It ist seen from this studies, that weak results may be wrong by a factor of 1000 ore even more. A discrepancy of a factor of three (or one third) may be classified as a good result!

All these formulae were derived from data measured under stationary flow conditions. Additionally the bed was completely movable. Thus the result of those formulae is always what is called the transport capacity and is only valid in stationary flow. The transport capacity is the amount of transport which will appear in the case of fully developed equlibrium conditions. It will therefore overestimate the transport rates, if the bed is only partially movable. For the case of instationary flow conditions no special formulae are available.

4 Entrainment/Settlement formulae

Entrainment formulae do not calculate equilibrium condition. Their result is what also is called the pick-up-rate. In other words it is the mass loss of the bed per unit area and unit time, if no inflow of sediment and no settlement is present. The entrainment results in the development of a concentration profile. Entrained sediment will then settle down an be entrained again. The amount of settled sediment is dependent on the concentration profile and on the properties of flow and sediment. Some formulae are available to calculate the settled sediment.

In tidal flows, where accelerations are not dominant, the flow may approximately be divided into time steps and each of them may be handled as quasi stationary. This works as was demonstrated by a number of tests.

5 Significant flow conditions

In tidal waters the tides change in a 14 day rythm between neap and spring tide as well as in longer cycles. As it is not possible (today) to compute for example a whole year or even longer periods in morphodynamic models tide by tide, it is an idea to specify a „morphodynamic“ tide. This is a tide, which, if for example is repeated for 14 days will produce (more or less) the same bed changes as the real neap-spring cycle. Based on such a tide the creation of the input data to drive the model is less extensive.

The author started to develop a 2d-morphodynamical model in 1992 (2). As found by the author, the bed elevations produced by a mean tide are very close to those produced by spring tide. This was tested for different natural systems. The main difference if mean tide or spring tide is used,is the time needed to produce similar bed changes. The time needed is longer when using the mean tide.

Similar findings were published by Latteux in 1995 based on 1d-computations (1). Latteux divided the tides of a 19 year tidal cycle into 20 tidal classes. Then he computed the transport rates (not the bed changes) devided into flood- and ebb transport for 1 year for all classes and added the result of each class multiplied by the probability of occurence. Then the transport rates for each class alone were computed for 1 year and compared to the previous results. It was found that a the transport rates of a tidal class between mean tide and spring tide are most close to that for the whole variety of tidal classes. A combination of a spring tide and a neap tide gave slightly better results. Although the transport-representative conditions are not necesserely exactly the morphodynamic representative results, this result may be good as a first choice.

6 Morphodynamic time scale

If only the flow conditions are a matter of interest, only a small number of tides has to be computed. In the case of morphodynamic modeling there is an interaction between flow, sediment transport and bed changes. Therefore, to model for example the period of one year, 706 tides have to be run. Up to now this is impossible in most cases. Thus to be able to compute longer time periods, one has to introduce a morphodynamic time scale, which is different to the hydrodynamic time scale. One of such methods is to multiply the bottom changes by a factor. The range of possible factors depends on the model used as well as on the hydrology and of the sedimentology of the investigated area and therefore are a matter of experience. Extensive tests demonstrated, that this method works very good within a variety of factors. This method is used by Zanke (1993, 1994, 1995) and by Latteux 1995.

7 Literature

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Cross-Shore Sediment Transport
- Comparison of Basic Approaches and Limitations -

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Abstract

The paper gives a short overview on the different modes of sediment transport in the inshore region of a sandy beach and presents a conceptual model for suspended sediment transport outside and inside the surf zone for engineering purpose. The nearshore sediment transport modelling is discussed with special emphasis on cross-shore transport under waves and on beach profile evulotion.

1 Introduction

The features of sandy coastlines change continuously with changing sea conditions and are referred to as coastal processes. The reason for the dynamic behaviour of sandy beaches and nearshore underwater topography is the concentration of wave energy onto a narrow strip of shoreline.

For each sea condition the underwater profile tries to reach an equilibrium form, such that the applied fluid forces acting on a grain are balanced by its resistance to motion. Due to the continuously changing sea conditions the geometry of a sandy shoreline is subject to a continuous change. The beach may erode or build up, sand banks form and disappear. Many of these changes are small and may go unnoticed, but every now and then, large changes occur, particularly due to severe storms when the shoreline may recede 20 or 30 m in just a day.

All changes have to be related to a time scale. Averaged over the years the beach may show trends to accretion or erosion. Superimposed on the trend are the short term changes due to changing wave conditions. In the absence of a trend the short term changes balance over a period of a year or more and the beach is said to be in dynamic equilibrium. A trend will lead over years to a change of the shoreline but the major long term changes in the shorelines have been caused by the changing mean sea level. The changes also have to be related to a spatial scale, a sand grain, a ripple, a sand bank, a coastal region, etc.

The physical processes, which in complex interactions create the geometry of the coastal zone include the most complex aspects of hydrodynamics; the prediction of waves, orbital velocities, currents, the spatial distribution of wave energy conversion in shallow water, etc. The transport of sediment due to wave action differs from that by unidirectional current mainly in the entrainment of sediment. In a current only situation the current has to entrain and transport the sediment. Under wave action the sediment can be entrained and suspended by the orbital movements of waves, and this suspended sediment could be transported by the feeblest of a current. The analytical description of the process of sediment movement under wave action is, however, much more difficult because of the periodic nature of the fluid forces and the variability of wave characteristics in magnitude and in direction with time. It has to be emphasized that the movements of sediments, the coastal processes, cannot be treated separately from the hydrodynamic conditions. Coastal processes are intimately coupled to the local hydrodynamics. Because of the very difficult hydrodynamics, there is a tendency to "forget" about it and talk about sand or sediment movement.

Coastal processes constitute work. The energy for this work comes from waves and currents. The analytical description of waves in deep and shallow water is fairly complete. Likewise, the description of tidal and ocean currents. However, the hydrodynamics in the surf zone has sofar defied a rigorous analytical description. The wave energy flux varies in time and direction. The wave form changes with shallowing water. The breaking process, the energy conversion, is a function of beach topography and wave form. The wave energy is first converted into turbulence and finally into heat. The intensity of turbulence, which has a strong influence on sediment movement, varies with the type of breakers over the surf zone. The water movements in the broken and reforming waves in the surf zone are only describable with the aid of gross simplifications. The influx of wave energy into the surf zone leads to a complex system of wave-induced currents. Since the sediments in the coastal zone are usually mobilized by wave action, currents play a central role in coastal processes as the conveyors of water-sediment mixtures (RAUDKIVI, 1997).

2 Littoral Transport

The transport in the inshore region is subdivided into a shore parallel and shore normal transport, the littoral or longshore and the onshore offshore or cross-shore transport. The littoral transport and littoral current are major features of a surf zone. The shore parallel component of wave energy flux drives the littoral current which is superimposed on the oscillating orbital movements of water due to wave motion. The net velocities of the longshore current are usually low and do not on their own entrain sediment, but because the sediment is mobilized by orbital velocities and turbulence due to energy conversion, the longshore currents assume an important role as conveyors of the water-sediment mixture. The transport is usually subdivided into suspended and bed load transport. The bed load transport is dominant at very low transport rates when the grains are rolled along on the bed and is predominant seaward of the surf zone. The bed load transport becomes insignificant at high transport rates which is dominated by suspended load. Both transport modes are always present. The total transport is usually analysed in terms of the shore-parallel and the cross-shore transport.

3 Conceptual Model for Suspended Sediment Transport

3.1 Introduction

The physics of sediment transport along a coastline is one of natures more complex problems, particularly in the surf zone. Although, significant advances have been made with the description of water movement in a wave field, as well as with aspects of movement of sediment particles, there is for the engineering application little more available than crude empirical formulae e.g. the CERC-Formula, the BIJKER-Formula, etc. Most of the numerical models available rest on grossly oversimplified assumptions. These serve a useful purpose in studies of large scale problems, but are of little help with localised problems in the surf zone. The latter are hopelessly complex and suffer from both of the inability to describe the processes of nature and lack of data to verify the results. The general problem is made even more difficult due to the different transport processes outside and inside the surf zone.

Outside the surf zone the sediment transport processes are governed by the orbital velocity and any superimposed currents. In the surf zone the transformation of wave energy into turbulence significantly increases the sediment entrainment by the orbital velocities. The orbital velocities themselves are ill-defined and act together with currents, caused by the wave energy flux into the surf zone. These longshore currents can exceed velocities of 1 m/s. By significant wave action the turbulence in the breaker zone is capable of dispersing sand throughout the depth of water, under certain conditions even gravel.

The characteristics of the distribution of suspended sediment are well illustrated by the field data collected in an international study on the Black Sea coast (Kossyan et al., 1982). Outside the surf zone there is a well developed suspended sediment layer (on long time average), supported by the diffusion of turbulence from the bed boundary layer. This layer can adequately be described by

$$c = c_0 \cdot e^{-a \cdot z} \qquad (1)$$

where c is temporal mean concentration at elevation z, c_0 is a reference concentration at the bed and 'a' is a decay parameter for the concentration with elevation. The c_0-value is more sensitive to the wave conditions than the a-value. The upper part of the distribution contains fine sediments and constitutes a wash load (dispersed sediment). This distribution is mainly dependent on the availability of fine silts rather than on hydraulic parameters. Available data from experiments in the large wave flume with fine and medium sands indicate that both 'a' and c_0 can be reasonably correlated with the maximum orbital velocity at the bed.

For the breaker line Kossyan et al. showed that the sediment distribution, as well as of the grain size distribution are almost uniform throughout the depth. Further inshore a new suspended sediment layer develops at the bed. The data by Kana

(1979) illustrate that the concentration at a given elevation is not a function of the coastal current velocity in the surf zone, but is strongly dependent on the breaker form.

The purpose of this study is to present a semi-empirical method for suspended sediment transport for engineering application. The method rests on the assumptions that

- the sediment is entrained (made mobile) by the actions of the orbital velocity and superimposed turbulence, i.e. the longshore current is assumed to be much weaker than the orbital velocities, and
- the transport is in the form of advection of the water-sediment mixture by the local currents.

The available numerical methods allow a satisfactory determination of the wave-induced coastal currents, to which tide induced currents can be added. Thus, in principle, the sediment transport rate can be calculated for a unit width from

$$q_s(x) = \int_0^h c(z) \cdot U(z)\, dz \tag{2}$$

where U(z) is the mean coastal current velocity at elevation z and h is the local water depth. The total littoral transport is then $Q_s = \Sigma\, q_s(x)$ to a distance outside the breaker zone, where the littoral current velocity becomes negligible.

3.2 Transport outside the Surf Zone

Outside the surf zone the sediment is suspended by the orbital velocities and the suspension is maintained by the diffusion of turbulence from the bottom boundary layer (Dette and Raudkivi, 1992).

Measured data on the distribution of suspended sediment under wave motion over a more or less flat horizontal bed outside the surf zone were used to derive relationships for engineering application. The data were obtained from the Large Wave Flume (LWF) in Hannover with nature-like depths (4.5 m) and waves (Fig. 1 and 2). The measured sediment distributions were fitted by eqn. (1). The data for a grain size $d_{50} \approx 0.25$ mm (Raudkivi and Dette, 1992) are described by

$$\frac{a}{T} = 0.4 \cdot u_m^{-2} \tag{3}$$

and

$$c_0 = 150 \cdot \left(\frac{u_m}{T}\right)^2 \tag{4}$$

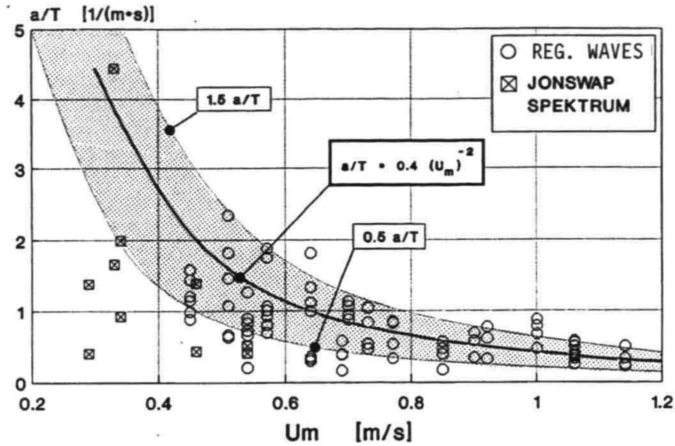

Fig. 1: Ratio a/T (decay parameter, wave period) in relation to the orbital velocity outside the surf zone (RAUDKIVI and DETTE, 1992)

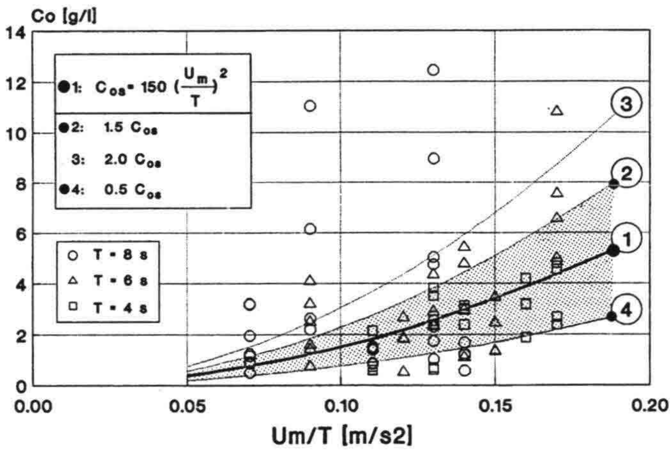

Fig. 2: Plot of bottom concentration c_0 in relation to the ratio u_m/T outside the surf zone based on measuring data from GWK (RAUDKIVI and DETTE, 1992)

It is suggested that for other grain sizes eqn. (3) be multiplied on the right hand side by (w/w_r) and eqn. (4) by $(w_r/w)^2$, where w_r is the fall velocity of the 0.25 mm grains. With the aid of these relationships the suspended sediment transport rate can be evaluated from eqn. (2). Fig. 3 shows a comparison of equ. (3) with similar approaches by Nielsen (1986) and Bosman et al. (1987). For a superim-

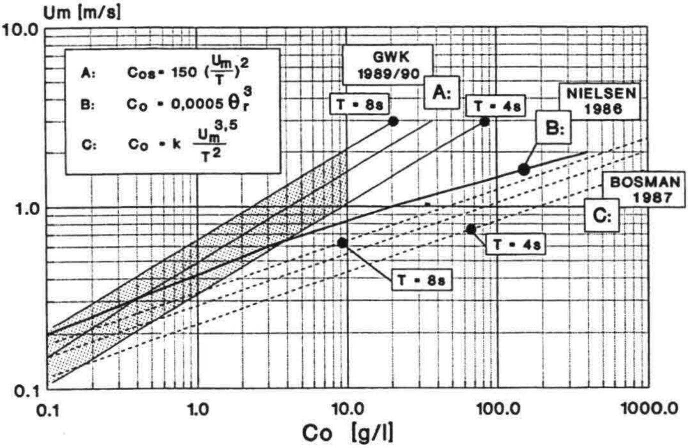

Fig. 3: Comparison of approach (eqn. 4) based on experiments in GWK with approaches by Bosman et al. (1987) and Nielsen (1986) - RAUDKIVI and DETTE, 1992

superimposed current, with logarithmic velocity distribution this leads to a $q_s(z)$-distribution with zero at the bed and a maximum close to the bed. Above the maximum the sediment transport rate decreases rapidly and with a distribution of the net velocity (together with a boundary layer) can become 'negative' quite near the bed, i.e., the transport direction can change.

3.3 Transport inside the Surf Zone

The hydrodynamics of the surf zone are extremely complex. Large amounts of wave energy are dissipated over a relatively small area through a multitude of breaker forms. The wave energy is first converted into turbulence and the transformation into heat is affected by viscosity at the dissipation scale of turbulence. Very little is known about the turbulence structure in the surf zone (Dette and Raudkivi, 1991).

The energy dissipation in the surf zone is an acute research topic and the literature on this topic is substantial. At the present state of knowledge of the structure of turbulence in the surf zone the assumption of a constant diffusion coefficient at the vicinity of the bed, as implied by eqn. (1), seems justified, i.e. if the simultaneous description of dispersed sediment is excluded. The dispersed sediment (generally less than 60 μm) is analoguous to the washload in rivers, i.e. dependent on availability and not on hydraulic conditions. The aim is to relate the parameters c_o and a with the local dissipation of wave energy. This would enable the calculation of the suspended sediment load in the surf zone with the aid of empirical relationships for c_o and a without any other assumptions or assumed parameters. The scatter of measured values of suspension is generally so great that a more 'refined' distribution than that by eqn. (1) is difficult to justify. Several numerical models are available which enable the calculation of energy dissipation over a given profile of the surf zone for applied purposes with adequate accuracy. The local dissipation can be related, with certain assumptions, to a nominal velocity of turbulence q' as

$$q' = K \cdot (\frac{D}{\rho})^{1/3} \tag{5}$$

where the coefficient K is approximately one, D is the dissipation (Watt/m^2) and ρ is the density of water. The power, P, required to suspend a grain of diameter, d, and mass, M, is

$$P \geq M \cdot g \cdot w \propto \rho_s \cdot g \cdot d^{3.5} \tag{6}$$

since the fall velocity of the grain w is approximately proportional to $\sqrt{d}$. For a given power the number of grains in suspension will increase rapidly and the fall velocity of the individual grains decreases, i.e. less power is required for the same total mass.

The dissipation of wave energy across the surf zone was measured in the LWF for spilling as well as plunging breakers. The distributions of suspension were measured with the aid of a suction harp (Bosman et al., 1987) and fitted by eqn. (1). The values of a and c_0 given by the measured data were correlated with the local energy dissipation D(x). The data (Raudkivi and Dette, 1993) show appreciable scatter but in a first approximation 'a' and c_0 were described by

$$a = 7 \cdot T \cdot D^{-0.7} \tag{7}$$

and

$$c_0 = 0.08 \cdot (D \cdot T)^{0.55} \tag{8}$$

where T is wave period in seconds and D in Watt/m^2 (Fig. 4 to 6). The scatter of the a/T values is within the 0.67 range if a few unlikely values are omitted and 80% of the c_0 values lie in the $\pm$ 0.67 range. Other forms of correlation of the data are being investigated. The empirical values of a and c_0 define the suspended sediment distribution for given wave conditions and in combination with the distribution of the net current velocity the longshore transport of suspended sediment can be calculated. The experiments were carried out using d_{50} = 220 µm and 330 pm. However, the available data do not enable an interpretation of the effect of grain size.

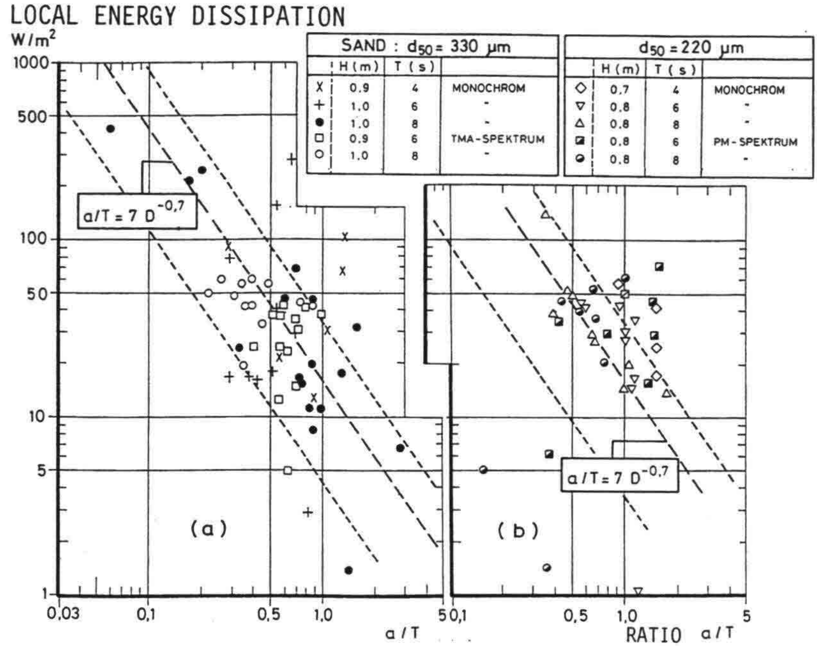

Fig. 4: Plot of ratio a/T (measuring data from GWK) in relation to the local energy dissipation D (RAUDKIVI and DETTE, 1993)

a) 1991 Test series with 330 μm sand
b) 1990 Test series with 220 μm sand
(dashed lines indicate width of ± 67 % Deviation)

4 Nearshore Sediment Transport Modelling

A literature review on cross-shore transport under waves and on beach profile evolution modelling is given by WU (1994). The summary of this review is cited in the following.

Two different approaches have evolved in the past decades for nearshore sediment transport modelling; one approach is to attempt to construct a sediment transport model from the basic mechanics on a refined scale and the other simply tries to relate gross sediment transport to averaged or main flow properties. The former is the classical approach of sorting out the mechanics of sediment mobilization and transport first on refined scales; transport models are then constructed at integrated temporal and spatial scales. This approach can, therefore, be referred to as mechanics approach or fine-scale approach. This type of model could yield better temporal and spatial resolution but may not be

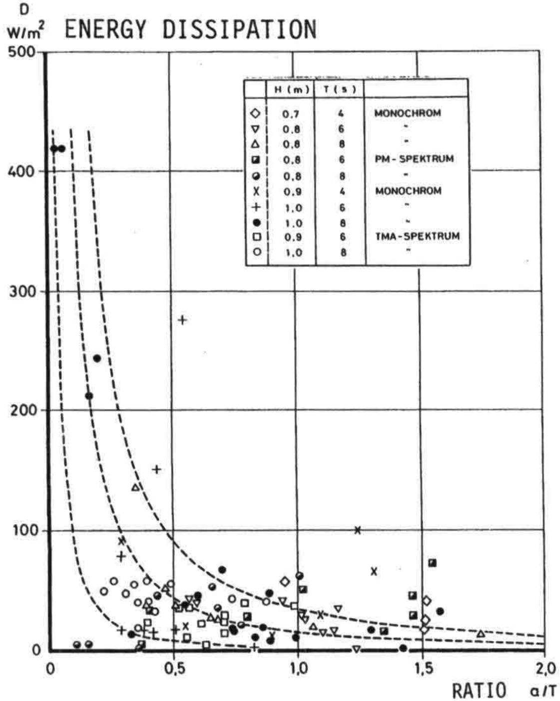

Fig. 5: Plot of a/T values in relation to the local energy dissipation (dashed lines indicate width of ± 67 % deviation - RAUDKIVI and DETTE 1993 -

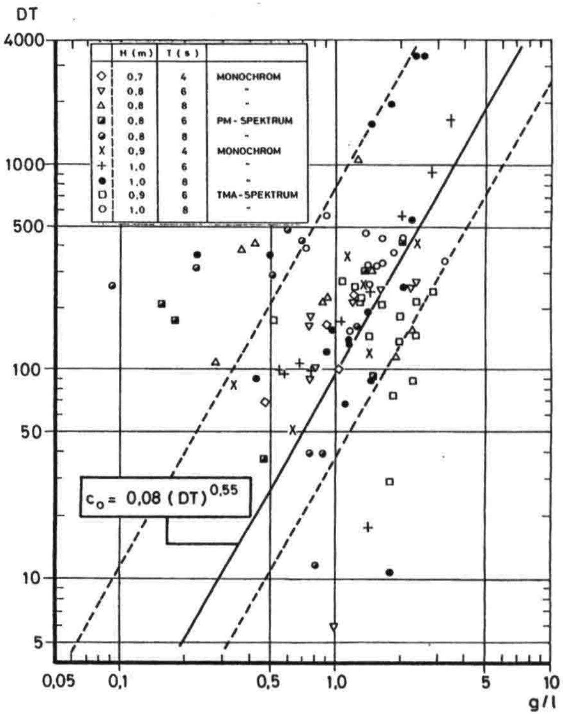

Fig. 6: Relation between bottom concentration c_o and product of D · T (dashed lines indicate width of ± 67 % deviation) - RAUDKIVI and DETTE (1993) -

practical for long term or large scale simulation. The other approach avoids the details of the mechanics of sediment transport and simply attempts to relate sediment transport to flow properties on a macro scale. The commonly inferred flow properties are temporal and spatially averaged wave energy, wave energy flux or rate of wave energy dissipation. Sediment transport models of this kind ignore the details and bypass the basic sediment mechanics; they are, more or less, heuristic and mainly based upon physical reasoning and/or empirical evidence. This type of approach is referred to as heuristic approach or macro-scale approach. Since models of this type deal with integrated flow properties they are usually more suitable for long term simulation.

The sediment transport models are then applied to the equation of mass conservation to model profile response.

As discussed, most of the cross shore profile evolution models are composed of four parts; (1) input and output unit; (2) hydrodynamic modelling; (3) sediment transport modelling and (4) morphological modelling. This is shown in Fig. 7. In the hydrodynamic modelling, the primary concern is the wave deformation, particularly the wave decay in the surf zone and associated changes in wave

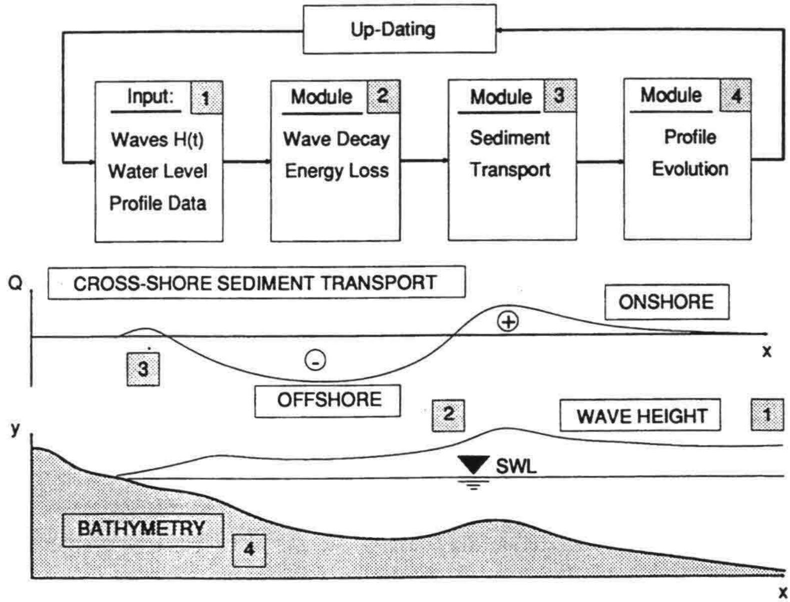

Fig. 7: Basic structure of cross-shore profile change models (WU, 1994)

energy and currents. In sediment transport modelling the main concerns are the cross-shore sediment transport rate and transport direction. There are a number of formalisms such as onshore/offshore transport, suspended-load/bed-load transport, current/wave transport. In morphological modelling the mass conservation equation is almost universally used to give the updated profile changes.

Most of the models can only accommodate regular wave input. A few of them could incorporate certain features of random waves, but only on time-averaged basis. The randomness of waves is considered by calculating time-averaged wave height transformation, the sediment transport related hydrodynamics and profile evolution will be determined with such wave deformation. Therefore, the randomness in wave is not directly coupled with sediment transport and profile evolution.

A list of general criteria for developing numerical profile was given by Dally (1980) for model evaluation: (1) The ability of generating general profiles of both normal and storm types depending on the wave conditions and sediment characteristics. (2) The ability of predicting the proper shape of these profiles; i.e. (a) the normal should be monochromatic and concave upwards and (b) the bar(s) of the storm profile should have the proper spacing and shape. (3) The ability of correctly predicting the rate of profile evolution. (4) The ability of responding to changes in water level due to tides, storm surges, or long term fluctuations. (5) The stability of the model is judged by its tendency of approaching a stable profile asymptotically if all the relevant parameters are held constant. An asymptotically stable profile is a profile approaching equilibrium and the gradient of the sediment transport in the on/off shore direction is approaching zero.

5 Acknowledgements

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DRBEM Analysis of Combined Wave Refraction and Diffraction in the Presence of Currents

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Abstract

In this paper, a more effective numerical model using the dual reciprocity boundary element method (DRBEM) is presented to analyze the combined effects of wave refraction-diffraction and currents. Firstly, The mild-slope equation with current effects is rearranged to an inhomogeneous Helmholtz equation. Applying the ordinary boundary element method to this inhomogeneous equation, the inhomogeneous part will result in a domain integral. It will make the computation complicated and messy. To improve this drawback, the DRBEM is adopted and thus taking the domain integral to the boundary. To verify the validity of this model, the relative amplitude around a circular island without current effects was firstly calculated, and compared with analytical solutions of Homma(1950). The agreement is fairly satisfactory. The combined effects of wave refraction-diffraction and currents around a circular island over a seabed with a variable depth are calculated by this model. The results show that the model has a great potential to be used to solve the combined wave refraction-diffraction and currents problem.

1 Introduction

The study of the combined effects of wave refraction-diffraction and currents becomes an important topic on ocean engineering nowadays. It has drawn attention of researchers to do the relate investigation. Homma(1950) used shallow water equation to investigate wave refraction and diffraction around a circular island. Berkhoff(1972), and later Vanstano and Reid(1975) derived the well known mild-slope equation to describe wave refraction and diffraction. Booij(1981), Liu(1983) and Kirby(1984) extended the mild-slope equation to include the effects of current. Based on the equation derived by Kirby(1984), a model using finite/boundary element method was proposed by Lin et al.(1994) to study the effects of wave refraction-diffraction together with a current. The model needs to place with a large number of finite elements in variable seabed domain. Thus, the computer storage and time are formidable.

Zhu(1993a), Zhu and Zhang(1995) developed a numerical model using the DRBEM, which was first proposed by Nardini and Brebbia(1982), to study the combined refraction and diffraction of water waves propagating around islands or solid offshore structures over a variable seabed. More recently, Hsiao et al.(1997) adopted the DRBEM to improve the numerical model combined perturbation and boundary element method proposed by Rangogni(1988) for wave refraction and diffraction problems. The results of them show that a considerable improvement in terms of numerical efficiency has been achieved with the adoption of the DRBEM. Therefore, the DRBEM is used in this study in order to provide a more effective numerical model dealing with the combined effects of wave refraction-diffraction and currents.

2 Mathematical Formulation

As shown in Fig. 1, the domain of interest is divided into two regions: a bounded region D_1 with a variable water depth, and a semi-unbounded region D_0 having a constant water depth. While in the first region, current together with a large obstacle are present. In the latter, it is assumed that effects of currents are negligible. The mild-slope equation derived by Booij(1981), Liu(1983) and Kirby(1984), describes the effects of wave refraction-diffraction in the presence of currents:

$$\frac{D^2\Phi}{Dt^2} - \nabla(CC_g\nabla\Phi) + (\sigma^2 - k^2CC_g)\Phi = 0 \ , \quad \text{where} \frac{D}{Dt} = (\frac{\partial}{\partial t} + \vec{U}\cdot\nabla) \tag{1}$$

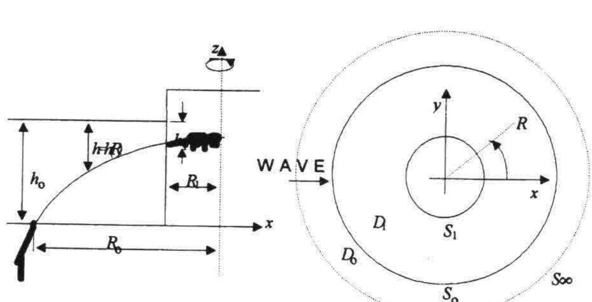

Fig. 1 Definition of computational domain

Here, $\vec{U}$ is the current velocity, ∇ represents a horizontal gradient operator, σ is the relative frequency, and k is the wave number. C and C_g are respectively the local phase and group velocities calculated from frequency σ and wave number k. For monochromatic waves with frequency ω one has

$$\Phi = \phi e^{-i\omega t} \tag{2}$$

Substituting Eq. (2) into Eq.(1) one can obtain

$$\nabla^2\phi + k_0^2\phi = \left[(k_0^2 - k^2) + \frac{1}{CC_g}(\sigma^2 - \omega^2)\right]\phi - \frac{1}{CC_g}(\nabla CC_g + 2i\omega\vec{U})\cdot\nabla\phi \tag{3}$$

Where k_0 is the wave number of constant water depth, ω is the absolute frequency, $i = \sqrt{-1}$ is the imaginary constant, ω and σ are related by Doppler's relation: $\omega = \sigma + \vec{k}\cdot\vec{U}$.

For waves propagating in the semi-unbounded region of constant water depth, without current interfering, Eq. (3) is then reduced to the Helmholtz equation

$$\nabla^2\phi_0 + k_0^2\phi_0 = 0 \tag{4}$$

Assume that the velocity potential ϕ_0, in the semi-unbounded region, can be expressed as a linear combination of the incident and scattered wave potential, i.e. $\phi_0=\phi^i+\phi^s$. Substituting the expression of ϕ_0 into Eq. (4), the governing equation for the scattered wave potential ϕ^S in semi-unbounded region D_0 can be obtained.

$$\nabla^2\phi^s + k_0^2\phi^s = 0 \tag{5}$$

In the bounded region D_1, the wave-current field around the obstacle is governed by Eq. (3), which is expressed as follow:

$$\nabla^2\phi_1 + k_0^2\phi_1 = [(k_0^2 - k^2) + \frac{1}{CC_g}(\sigma^2 - \omega^2)]\phi_1 - \frac{1}{CC_g}(\nabla CC_g + 2i\omega\vec{U})\cdot\nabla\phi_1 \tag{6}$$

where, ϕ_1 denote the velocity potential corresponding to the variable water depth in the bounded region.

The boundary conditions for the computational domain are now specified as follows:

1. The boundary condition along the surface of the obstacle S_1 is given by

$$\frac{\partial\phi_1}{\partial n} = 0 \tag{7}$$

2. In the semi-unbounded region, the scattered wave potential ϕ_0 must satisfy the Sommerfeld radiation condition

$$\lim_{r\to\infty}\sqrt{r}\left(\frac{\partial}{\partial r} - ik_0\right)\phi^s = 0\ , \quad \text{where } r = \sqrt{x^2 + y^2} \tag{8}$$

3. For the boundary between bounded and semi-unbounded region, conservation of mass and energy flux of the fluid leads to the requirement that

$$\begin{cases} \phi_1 = \phi_0 = \phi^s + \phi^i \\ \dfrac{\partial\phi_1}{\partial n} = -\dfrac{\partial\phi_0}{\partial n'} = -\left(\dfrac{\partial\phi^s}{\partial n'} + \dfrac{\partial\phi^i}{\partial n'}\right)\ , \quad \vec{n}' = -\vec{n} \end{cases} \quad \text{on } S_0 \tag{9}$$

here, n and n' are respectively the outer normal derivative for the bounded and semi-unbounded region

3 Description of the model

In semi-unbounded region D_0, the governing equation for the scattered wave potential is Eq. (5), the Helmholtz equation. If the weighting function ϕ^* is chosen as $\phi^* = -\frac{i}{4}H_0^{(1)}(kr)$, where $H_0^{(1)}$ is the Hankel function of the first kind of zero order and r is the distance between a source point and a field point. By applying Green's second identity, and with the boundary condition on S_∞ (Eq. (8)) substituted into Eq. (5) leads to the following boundary integral equation :

$$C_{0i}\phi_i^s = \int_{S_1}\left(\phi^s\frac{\partial\phi^*}{\partial n'} - \phi^*\frac{\partial\phi^s}{\partial n'}\right)ds\ , \qquad C_{0i} = \begin{cases} 1 & \text{in } D_0 \\ \frac{1}{2} & \text{on } S_0 \end{cases} \tag{10}$$

In the bounded region D_1, the governing equation for the wave-current field is, Eq. (6), an inhomogeneous Helmholtz equation. Multiplying both side of Eq. (6) by the same weighting function ϕ^* as that in Eq. (10) and applying Green's theorem again one can obtain

$$C_{1i}\phi_{1i} - \int_S \phi_1\frac{\partial\phi^*}{\partial n}ds + \int_S \phi^*\frac{\partial\phi_1}{\partial n}ds = \iint_{D_1}\phi^* b\,dA\ , \quad C_{1i} = \begin{cases} 1 & \text{in } D_1 \\ \frac{1}{2} & \text{on } S \end{cases} \tag{11}$$

where $S = S_1 + S_0$ is the boundary of region D_1, and b denote the inhomogenous part, i.e., the right hand side of Eq. (6)

The system of integral equations Eq. (10) and Eq. (11) can be simplified once the boundary conditions, Eq. (9), on the boundary S_0 are imposed. Therefore, Eq. (10) and (11) can be combined as:

$$\left(C_{1i} + C_{0i}\right)\phi_{1i} - \int_{S_1}\left(\phi_1\frac{\partial\phi^*}{\partial n} - \phi^*\frac{\partial\phi_1}{\partial n}\right)ds - \int_{S_0}\left(\phi^i\frac{\partial\phi^*}{\partial n} - \phi^*\frac{\partial\phi^i}{\partial n}\right)ds = C_{0i}\phi^i + \iint_{D_1}\phi^* b\,dA \tag{12}$$

The domain integral on the right hand side of Eq. (12) is now dealt with by DRBEM. Based on the theory of DRBEM (Partridge, Brebbia & Wrobel, 1992), the function b can be approximated as:

$$b = \sum_{k=1}^{N+L}\alpha_k f_k \tag{13}$$

where, N is the number of nodes on the boundary S, L is the number of internal points, α_k are unknown coefficients to be determined by the collocation method, and f_k are expansion functions, which can be expressed by the distance function r_k , from the source point to a specified point and can be chosen as $f_k=1+r_k$. If there exist a function $\hat{\phi}$ that satisfies (Zhu, 1993b)

$$\nabla^2\hat{\phi}_k + k_0^2\hat{\phi}_k = f_k \tag{14}$$

Here the particular solutions $\hat{\phi}_k$ for Eq. (14) can be found in Zhu(1993b). By substituting Eq. (13) and (14) into Eq. (12), and can be rewritten as:

$$(C_{1i}+C_{0i})\phi_{1i}-\int_{s_1}\left(\phi_1\frac{\partial\phi^*}{\partial n}-\phi^*\frac{\partial\phi_1}{\partial n}\right)ds-\int_{s_0}\left(\phi^i\frac{\partial\phi^*}{\partial n}-\phi^*\frac{\partial\phi^i}{\partial n}\right)ds$$

$$=C_{0i}\phi^i+\sum_{k=1}^{N+L}\alpha_k[C_{2i}\hat{\phi}_{ik}-\int_s\hat{\phi}_k\frac{\partial\phi^*}{\partial n}ds+\int_s\phi^*\frac{\partial\hat{\phi}_k}{\partial n}ds] \tag{15}$$

After discretizing the boundaries S_0 and S_1 and applying the standard DRBEM procedures, a linear system of algebraic equations is obtained as

$$-[H_{ij}][\phi_{1j}]+[G_{ij}][q_{1j}]-[\overline{H}_{ij}][\phi_j^i]+[\overline{G}_{ij}][q_j^i]=\sum_{k=1}^{N+L}\{-[H'_{ij}][\hat{\phi}_{jk}]+[G'_{ij}][\hat{q}_{kj}]\}[\alpha_k] \tag{16}$$

where $q_{1j}=\frac{\partial}{\partial n}\phi_{1j}$, $q_{jk}=\frac{\partial}{\partial n}\hat{\phi}_{jk}$, $\alpha_k=[f_{ik}]^{-1}\cdot b$, and $[H_{ij}]$, $[G_{ij}]$, $[\overline{H}_{ij}]$, $[\overline{G}_{ij}]$, $[H'_{ij}]$ and $[G'_{ij}]$ are of the usual meaning. While the boundary conditions on S_1, Eq. (7), are imposed, Eq. (16) can be solved with the inversion of an (N+L) by (N+L) complex matrix.

4 Numerical examples

Table 1 Conditions used for numerical calculation

	R_1(m)	R_0(m)	h_1(m)	h_0(m)	h_0/h_1
Topography A	10,000.00	11,547.00	3,000.00	4,000.00	1.33
Topography B		17,329.18	1,333.33		3.00
Topography C		20,000.00	1,000.00		4.00

Homma's (1950) analytical examples for wave refraction-diffraction around a circular island over a paraboloidal shoal (as shown in Fig. 1) have been adopted as cases discussed in this paper. Three topographies (A, B and C) are considered as listed in Table 1. The wave period T=240 sec, 480 sec, 720 sec and 1440 sec are chosen as the conditions of the incident waves. To verify the validity of this model, the relative amplitude around a circular island on a paraboloidal shoal, without current effects (i.e. U=0) was firstly calculated. Fig. 2 show the numerical results for three topographies together with the analytical solutions of Homma (1950). The agreement is fairly satisfactory.

Variations of the relative amplitude around a circular island under the effects of current are calculated for topography A and B. The current velocities U are 10 m/sec and 20 m/sec, the intersection angle between wave and current varies from 0°, 90° to 180°. The numerical results of the relative amplitude around the circular island are shown in Figs.3 ~ 5 for topography A. From these figures, it demonstrates that, (a) with waves propagating on an a favorable current, the relative amplitude becomes smaller as compared with the case of absence of current, and also decreases with increasing current velocity, and (b) for waves propagating on an adverse current, an opposite trend is found. Figs. 4 shows the variation of the relative amplitude around the island for the angle between wave and current is 90°. In the downwave region behind the island, the relative amplitude in the outflow side (circular angles equal to 300° ~ 360°, the circular angle measured clockwise from the positive x direction are defined.) is larger than the case of zero current and increases with increasing current velocity, the opposite trend can be detected in the inflow side (circular angles equal to 0° ~ 60°).

5 Conclusion

The following conclusions can be drawn from this study:

(1) The dual reciprocity boundary element method is adopted to develop a more effective numerical model for wave-current and structures interaction problem. The cases with no current effects was firstly calculated, and compared with the analytical solutions of Homma(1950). The agreements are satisfactory. Using this model, the combined effects of refraction-diffraction and currents around a circular island were further calculated. The numerical results show qualitative agreement. Therefore, the DRBEM model proposed here has its adaptability to deal with the interaction between wave-current and large structures.

(2) With waves propagating on a favorable current, the relative amplitude around a circular island over a varying topography becomes smaller as compared with the case of absence of current, and also decreases with increasing current velocity. However, for waves propagating on an adverse current, an opposite trend is found.

(3) In the downwave region behind the island, the relative amplitude in the outflow side is larger than the case of zero current and increases with increasing current velocity, whereas, the opposite trend can be detected in the inflow side.

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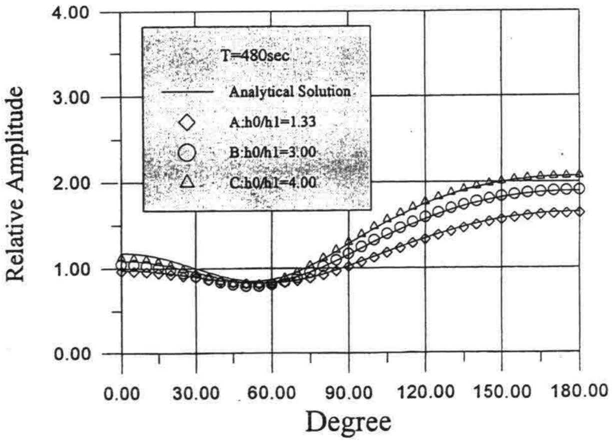

Fig. 2 Comparison between analytical solutions of Homma(1950) and present numerical results for the relative amplitude around a circular island.

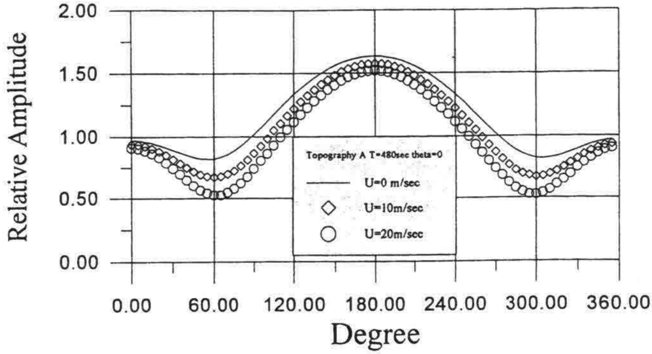

Fig. 3 Relative amplitude around a circular island (topography A) for the intersection angle θ=0°.

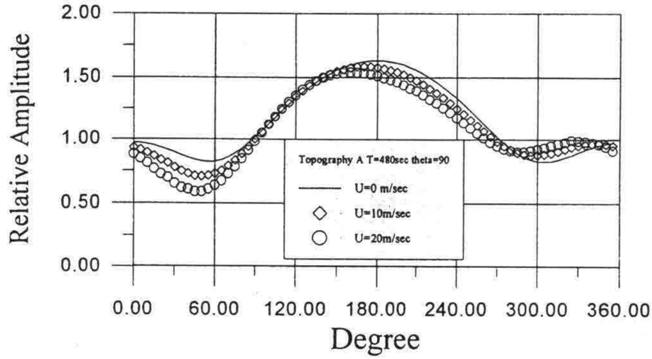

Fig. 4 Relative amplitude around a circular island (topography A) for the intersection angle θ=90°.

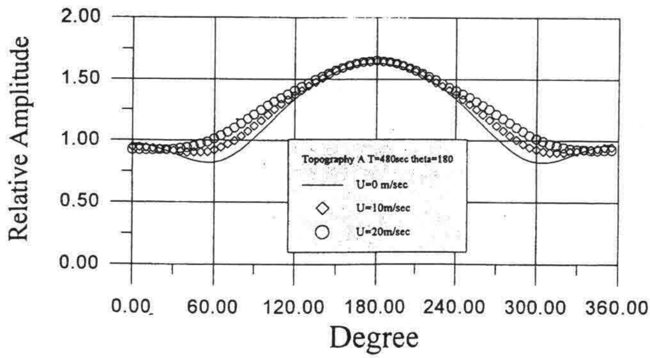

Fig. 5 Relative amplitude around a circular island (topography A) for the intersection angle θ=180°.

A Numerical Model for Non-linear Interaction of Water Waves with Submerged Obstacles

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Abstract

In this study we solve numerically the full Navier-Stokes and continuity equations with a finite difference method to investigate the effects of flow separation induced by the propagation of time-periodic water waves over a submerged rectangular obstacle. Water surface elevations and fluid velocities were obtained for a range of non-breaking wave conditions. Performance of the numerical model is accessed by comparing the numerical computational results with available experimental measurements. The computed wave profiles at various locations and local flow fields around the submerged obstacle agree favorably with experimental observations. The results indicate that the formation and growth of separation region respond directly to the wave transformation over the submerged obstacle, causing a variety of different eddy patterns.

1. Introduction

The propagation of surface gravity waves over a submerged rectangular obstacle has been extensively examined experimentally and theoretically because of its practical importance in oceanography as well as in coastal engineering. The flow owing to surface gravity waves over a submerged structure subjects to the effects of wave nonlinearity resulting from the increased ratio of wave height to water depth over the obstacle and fluid viscosity near the sharp corners. It therefore makes theoretical analyses rather difficult. This work indicates a numerical study is conducted by solving the Navier-Stokes equations for incompressible viscous fluid in investigating the propagation of waves over a rectangular submerged obstacle and in better understanding of the flow field of waves over a barrier.

Previous theoretical studies on the flow owing to surface waves for a given incident wave condition over an obstacle geometry were generally treated using potential theory which assumes that the flow is irrotational. Within the framework of a linear theory, the best known and most often quoted result originates from Lamb(1932,Art. 176) and extended by Jeffreys (1944). Nonlinear interaction of water waves with a submerged rectangular obstacle has been studied recently by Rey et al (1992), Grue(1992) and Ohyama and Nadaoka(1993). According to laboratory experiments, Ting and Kim (1994) conducted laboratory experiments to investigate flow separation effects induced by the propagation of surface waves over a submerged rectangular obstacle. Fluid velocities and water surface elevation in the near field of the submerged obstacle were measured and the processes of vortex generation and wave deformation were observed. Comparing results from previous theoretical studies and experiments indicates that inviscid theory cannot accurately reproduce the experimental observations at the obstacle's corner edges. The significant discrepancies between theory and experiment are due to the presence of flow separation which cannot be modelled by potential theory. Herein, we perform a numerical simulation to calculate the flow field as well as the water surface deformation using a two-dimensional viscous equations for the laminar flow of water waves traveling over a submerged rectangular obstacle. The vortex formation and subsequent migration at different Ursell numbers are investigated.

2. Statement of the problem

The physical problem considered herein is a two-dimensional continuous periodic wave trains in a still water of depth, d, passing over a submerged rectangular obstacle with height, h_1, and width, b. The incident wave height and period are represented by H and T, respectively. Figure 1 presents the geometry and coordinates of the flow problem. The governing equations for continuity and momentum for unsteady incompressible viscous flows are

Continuity : $$\frac{\partial u}{\partial x}+\frac{\partial v}{\partial y}=0 \tag{1}$$

Momentum : $$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}=-\frac{\partial p}{\partial x}+g_x+\nu\left(\frac{\partial^2 u}{\partial x^2}+\frac{\partial^2 u}{\partial y^2}\right) \tag{2}$$

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}=-\frac{\partial p}{\partial y}+g_y+\nu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right) \tag{3}$$

Here, x and y denote the coordinates of a fixed Cartesian system; t is the time; u and v represent the velocity components in the x- and y-directions, respectively; p is the pressure divided by the constant fluid density; g_x and g_y denote the gravitational acceleration in the x- and y-directions, respectively, and ν is the kinematic viscosity of fluid.

The fluid configuration at the free-surface is defined by a volume-of-fluid (VOF) (Nichols,1975) function F(x,y,t), whose value is unity at any point occupied by the fluid and zero elsewhere. The equation governing the time evolution of F is

$$\frac{\partial F}{\partial t}+u\frac{\partial F}{\partial x}+v\frac{\partial F}{\partial y}=0 \tag{4}$$

which states that F moves with the fluid.

To make the problem computationally feasible, artificial confining boundaries are placed sufficiently far from the obstacle. The upstream boundary in front of the obstacle should be long enough to prevent the influence of wave reflection caused by the submerged obstacle. Non-slip boundary condition is imposed on the obstacle surface. A third-order Stokes' wave (Wiegel,1964) is employed for generating periodic wave trains in the upstream boundary. The free surface boundary conditions must satisfy the kinematic and dynamic conditions. In addition, a radiation condition must be imposed to avoid upstream waves at the downstream boundary. In the present study, the radiation condition of Orlanski(1976) was employed to eliminate the nonphysical wave reflection from downstream.

3. Numerical Method

A modified version of the marker-and-cell (MAC) method (Harlow and Welch,1965) is employed to obtain the numerical solution of equations (1) through (3). Staggered grid arrangements are employed in which velocity components are defined at the midpoints of the cell sides, and the pressure and volume of fluid function are defined at the center of the cell.

A time-dependent solution is obtained by advancing the flow field variables through a sequence of short time steps of duration δt. The advancement in one time step is calculated in two stages. First the velocity components are all advanced using the previous state of the flow to calculate the accelerations caused by convection, viscous stress, pressure gradients, etc. However, this explicit time advancement does not necessarily cause a velocity field with zero divergence. Thus, in stage two, adjustments must be made to insure mass conservation. This is achieved by adjusting the pressure in each cell in such a way that there is no net mass flow in or out of the cell. In cells containing a free surface the continuity equation is replaced by the dynamic condition of continuity of the normal stress. A change in one cell influences neighboring cells so that this pressure adjustment must be iteratively performed until all cells have simultaneously achieved a zero mass change. The last step of the computational cycle consists of advancing the F-function in time using Eq.(4). A detailed discussion of this topic can be found in the work of Nichols and Hirt(1980).

4. Presentation and Discussion of Results

The major objective in developing this numerical model is to investigate the wave transformation over a submerged obstacle by using the viscous flow equations. The flow field of a propagating wave around the submerged obstacle relies on the wave conditions and the length aspect ratios of the submerged object. In the subsequent computations the water depth and dimensions of the submerged object were held fixed. The wave deformation and the fluid kinematics in the vicinity of the submerged obstacle were examined by varying the incident wave height and wave period. The parameter of HL^2/d^3 , the Ursell number, is employed to represent the effect of wave nonlinearity. Table 1 lists the computational conditions of these flows.

4.1 Comparison with experimental measurements

To assess the accuracy of the governing equations and the validity of the associated computer program, the physical experiment conducted by Ohyama et al.(1994) is employed to verify the present numerical method. In the study of Ohyama et al, the experimental incident wave was conducted at a wave flume with 17m long, 0.4m wide and the water depth d was set at 25cm. The incident wave height and the period were 2.5cm and 1.018sec, respectively. The submerged rectangular obstacle had a width of 100cm and a height of 17.5cm. The wave profiles were measured at three locations : one at the center of the shelf, and the others 125cm from the center of the shelf on both sides. Figure 2 depicts the comparisons of wave profiles between the numerical calculation and the Ohyama et al's measurement at three locations are given. It can be seen that the computed profiles agree favorably with the corresponding experimental observations indicating that the current numerical model is sufficiently reliable.

4.2 Eddy motions induced by wave propagations

Figure 3 indicates the computed velocity field and water surface elevation in one wave period for the case that the water depth ratio $(d-h_1)/d$ is 0.6, and the Ursell number, $Ur=HL^2/d^3$, is 2.88. Wave trains propagate from the left to the right above the submerged rectangular obstacle. The velocity vectors in this figure, and others to follow, have length and directions indicating the magnitudes and directions of the velocity at the base of each vector. The fluid velocities are normalized with respect to the maximum horizontal velocity of the incident wave at still water level and the scale representing the normalized velocity is shown in each figure. In Case 1, the Ursell number is very small, and the nonlinearity of the incident wave is not significant. Figure 3 indicates that flow separates at the up-wave and down-wave edges of the submerged obstacle during the propagation of wave trains. There is a tendency of flow separation followed by the reattachment on the surface of an obstacle resulting in a localized separation "bubble" around each corner. The size of the separation "bubble" is small as compared to the dimension of the submerged obstacle. It can be seen that the flow separation modifies the fluid kinematics around the edges of the submerged obstacle, but it has no effect on the surface waves.

Figure 4 presents the computed flow field for Case 2 with d/L=0.102 and Ursell number being 13.4. The increase of the Ursell number enhance the vortex generation and shedding from the edges of the submerged obstacle. Fig. 4 indicates a periodic sequence of vortex motion resulting from the passage of wave trains over the submerged rectangular obstacle. Starting from the time when the crest of the wave is immediately above the up-wave edge of the submerged obstacle(t/T=0) it can be seen that flow separates from the up-wave edge and reattaches on the obstacle surface forming a separation "bubble". As the wave crest propagates downstream (t/T=1/8), the separation "bubble" diminishes in the decelerated flow. Meanwhile, the horizontal velocities under the wave accelerate to their extreme values at the down-wave edge and a clockwise vortex is generated at the obstacle's lee side. With the existing counter-clockwise vortex, an eddy pair is formed. For t/T=2/8 to t/T=3/8, the water surface above the submerged obstacle continues to fall throughout the shallow region, and the fluid enters the obstacle's lee side. In this part of the motion the eddy pair is observed to be strengthened. When the trough of the wave is approached (t/T=4/8), as Fig. 4 reveals the flow direction at the up-wave edge of the submerged obstacle has effectively reversed from that when the crest is above the obstacle. The separation "bubble" produces at the up-wave edge disappears and a recirculation zone with counter-clockwise vortex develops above the submerged obstacle at the down-wave edge. This separation bubble continues to grow as the wave trough passes (t/T=5/8 to t/T=6/8). At the up-edge, a counter-clockwise vortex is formed on the weather side, but it remains much weaker than the clockwise vortex at the obstacle's lee side. At t/T=7/8 the water surface above the submerged obstacle is rising, the horizontal velocities become positive and accelerating. The fluid separated from the down-wave edge does not reattach to the submerged object but comes up with the water surface. Once again a separation "bubble" is formed again at the up-wave edge, and the counter-clockwise vortex on the down-wave side is shed and swept back around the corner.

In Case 3, the ratio of water depth to wavelength is 0.082 and the Ursell number is 23.73. The nonlinear effect of waves become more significant. Figure 5 represents the computed water surface and flow field in one period of wave propagation. The separation zones formed at the up-wave and down-wave edges of the submerged obstacle are stronger than those of Case 1 and Case 2. The vortices on either side of the submerged obstacle are relatively complex in comparing to the previous cases. The eddy motion intensity increases with increasing Ursell number. The results indicate that the presence of separation and eddy motion may have greatly modified the velocity distribution near the obstacle for higher Ursell number.

5. Conclusions

This work presents a numerical model for the propagation of water waves over a submerged rectangular obstacle using the viscous flow equations. The subtle effects coming from the fluid viscosity are put into the presence of flow separation and vortex generation on the corner and in the vicinity of the obstacle.

For a given water depth and dimension of the submerged obstacle, detailed velocity fields and surface wave profiles are obtained for different wave conditions. When HL^2/d^3 is small the separated flow reattaches itself to the obstacle's surface and forms a small separation zone as compared with the dimension of obstacle. The presence of flow separation has no effect on the surface wave. As HL^2/d^3 increases the separated boundary layer forms a complete vortex on the side of the obstacle. Vortex pair sheds and forms at the obstacle's lee side. The vortex motion is more powerful and energetic at the obstacle's lee side than on the weather side.

ACKNOWLEDGMENT

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Table 1 Wave conditions and obstacle geometry

Variables	Case 1	Case 2	Case 3
d (cm)	25	25	25
b (cm)	15	15	15
h1 (cm)	10	10	10
h2 (cm)	15	15	15
H (cm)	2.5	3.5	4.0
T (ses)	1.018	1.650	2.000
L (m)	1.342	2.446	3.045
d/L	0.186	0.102	0.082
H/L	0.0186	0.0143	0.0131
$Ur=HL^2/d^3$	2.88	13.4	23.73

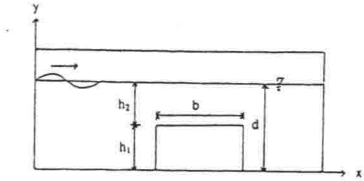

Fig.1 Geometry and coordinate of the flow problem.

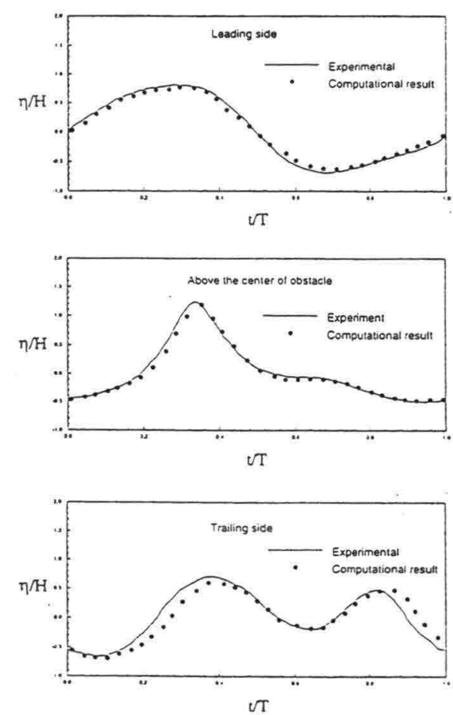

Fig.2 Comparison of computed wave profiles with experimental observations.

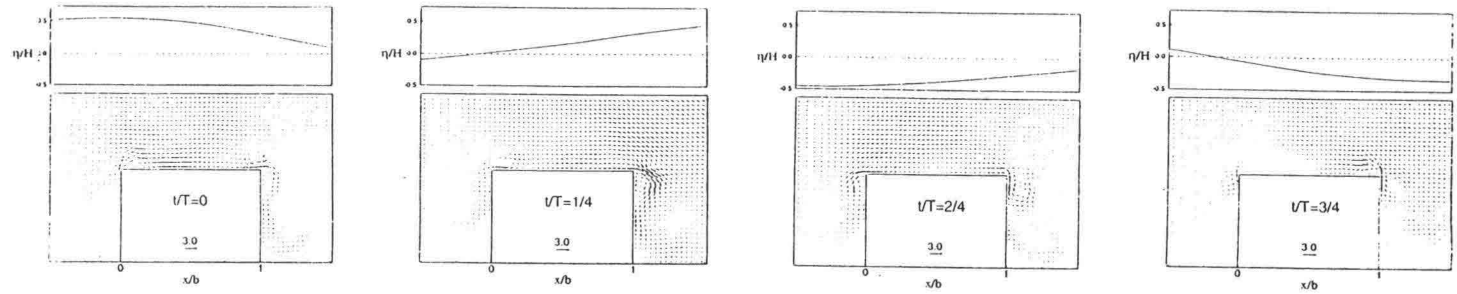

Fig.3 Velocity vectors and surface profile for Case 1; Ur=2.88.

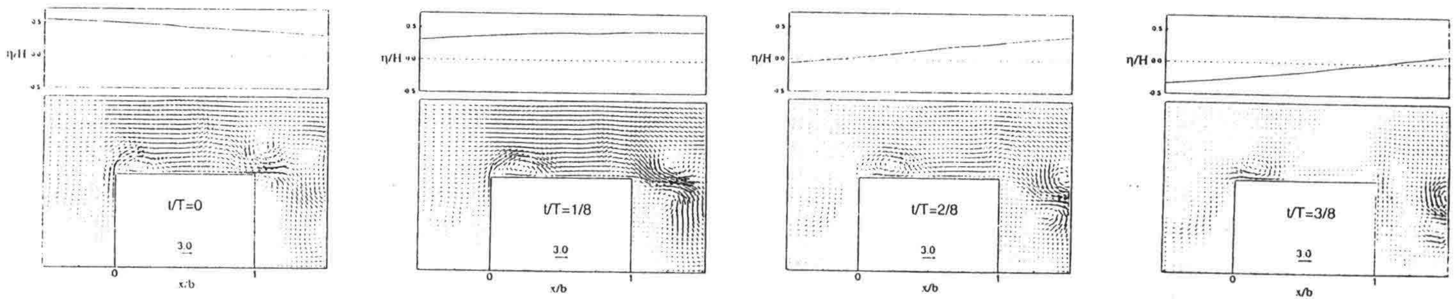

Fig.4 Velocity vectors and surface profile for Case 2; Ur=13.4.

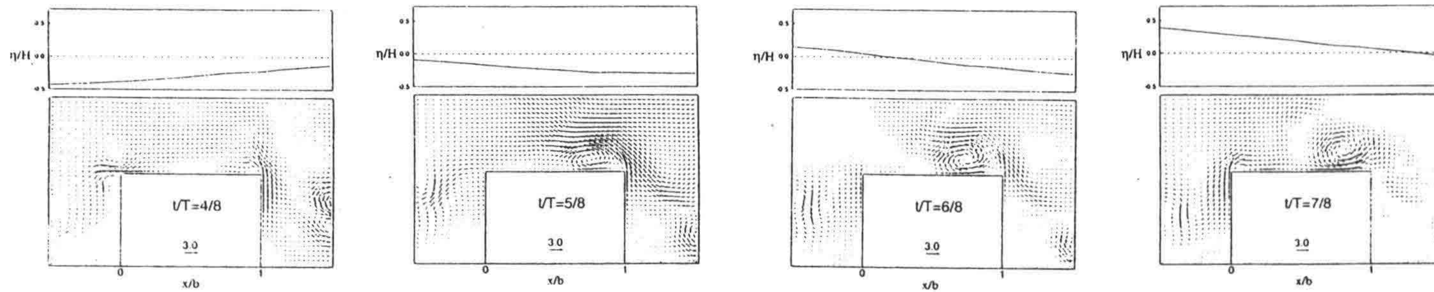

Fig.4 Velocity vectors and surface profile for Case 2; Ur=13.4.

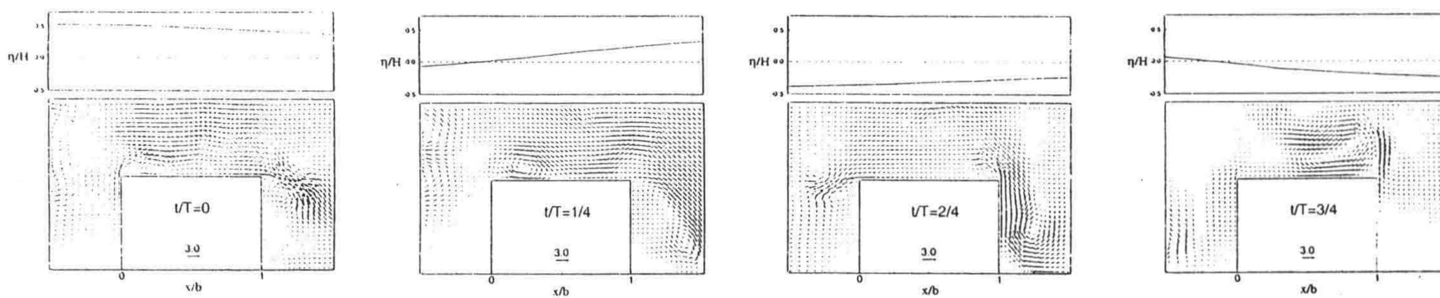

Fig.5 Velocity vectors and surface profile for Case 3; Ur=23.73.

Topic III

Simulation Methods

Coastal Processes, Waves and Currents

Chairman: Robert R. Hwang

Coupling of Numerical Wave and Current Models

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Abstract: Numerical models for wave and current simulations are becoming standard engineering tools for design of coastal works. The models have to simulate wind, wave, current and morphodynamic processes. The paper presents an appoach for coupled treatment of all these processes and discusses conditions under which decoupling might be possible. An example from application is given.

1 Introduction

The simulation of beach processes is becoming more and more an application field for numerical models. Though the processes are not yet fully describable by mathematical relations and are based on quite numerous empirical relations remarkable success has been achieved in engineering applications.

There are different numerical models around. The basic differential equations concern the physical phenomena of

- wind field and air pressure distribution,
- short wave propagation,
- current field and mean water-level variation,
- transport of sediment,
- sea bed evolution,
-

which all interact. The problem for the practizing engineer is to determine the right combination of equations, linearizations and splittings into submodels.

The paper deals with the simple approach of combining linear wave-theory based models with vertically integrated current models. Models of this kind are a compromise between non-linear Serre-type equations based models, which are very costly for large area and long-time simulations and bulk formula based empirical approaches.

The correctness of the chosen approach is difficult to validate as in nature individual waves from a spectrum of waves interact. However for engineering applications the reproduction of single waves is not that important as long as the behaviour of the whole system can be realistically simulated within reasonable accuracy. The accuracy depends on the purpose of application. The analysis of wave heights and periods is of concern for the interaction with structures. For beach stability the impact on the morphological evolution has to be considered.

Thus the whole set of differential equations has to be considered.

2 Interaction of Processes

The differential equations describing wave-current processes may be split into contributions based on energy, mass or momentum conservation as well as in terms standing for empirically determined relations. This fact allows for inspecting the degree of interdependencies of the state variables and splitting the full set of equations into individually treatable subsets of equations.

The following notation will be used. The orthogonal coordinate system x_i is supposed to coincide with the free surface of the water body at rest. The corresponding water depth is given by *d*. The deviation from this level of reference due to the current field is represented by η. Thus the surface waves travel over a water depth of $h = d + \eta$. The position of the free surface is given by $h + \zeta$.

2.1 Wind Model

The wind represents the driving force to waves and currents. It is generally accepted that there is no backward influence from the wave-current field to the wind field. Thus the wind and pressure field may be modeled seperately from all other water related state variables. Knowledge about the wind field is important for determining the wave properties which have to be prescribed at the boundaries of areas under investigation. Inside the modeling areas the distribution of the wind- and pressure fields may be considered as external forces driving the currents. Its impact on the formation, distorsion and propagation of waves is commonly neglected. This is acceptable due to the assumptions of linear theory which cannot directly account for those effects.

2.2 Wave Model

The wave model to be discussed here holds for monochromatic waves of small amplitude and steepness. It is formulated in terms of propagation of waves. It includes the effects of shoaling, refraction, diffraction and wave breaking. This formulation is appropriate for simulation of beach processes where reflections are small and may be neglected.

The field equations for the wave-number vector components K_i had first been given by HAYES.

$$K_{i,t} + \sigma_{,i} = 0 \tag{1}$$
$$K_{i,j} = K_{j,i} \tag{2}$$

together with the frequency dispersion relation σ for monochromatic waves

$$\sigma^2 = g\, k \tanh(k\, h) \tag{3}$$
$$k = 2\,\pi / L \tag{4}$$

In these equations k is the wave number, L is the wave length, h stands for the water depth at which the wave is propagating and g is the earth acceleration. The horizontal directions are marked by i = 1,2.

The solution for the wave number vector components (1,2) is difficult. There are three equations for two unknown. Equations (2) determines a rotation free vector field. For solution, this constraint may just be added to equation (1) as a Lagrange constraint, premultiplied with a factor λ.

$$K_{i,t} + \sigma_{,i} + \lambda \left(K_{i,j} + K_{j,i}\right) = 0 \tag{5}$$

The Lagrange factor λ has to be determined. MILBRADT shows by considering all terms containing the wave vector that a dimension correct consistent formulation is obtained if λ corresponds to the group velocity

$$C_g = \frac{1}{2}\left(1+G\right)\frac{\sqrt{\left(gk \tanh(kh)\right)}}{k} ; G = \frac{2kh}{\sinh(2kh)} \tag{6}$$

Equations (1) and (2) are thus substituted by

$$K_{i,t} + \sigma_{,i} - G_g \frac{K_j}{k}\left(K_{i,j} - K_{i,j}\right) = 0 \tag{7}$$

The derivation given here is quite general. It includes diffraction if the BATTJES relation is used which relates the wave number to the distribution of wave amplitudes a.

$$K^2 = k^2 + \delta \; ; \; \delta = \frac{1}{a} a_{,ii} \tag{8}$$

MILBRADT showed the consistent extension of the equations for systems with bottom slope. The frequency dispersion relation has to be extended into

$$\sigma = g\, k \frac{\tanh(kh) + s}{1 + s \tanh(kh)} \tag{9}$$

in which s represents the effect of bottom slope which depends on the wave amplitude a.

$$s = -\frac{1}{k}\frac{1}{A} A_{,i} h_{,i} \; ; \; A = -g \frac{a}{\sigma} \tag{10}$$

When waves are propagating on a current field with the vertically averaged flow velocity U means, the frequency dispersion relation σ has to be extended into σ_a which refers to a fixed frame of reference

$$\sigma_a = \sigma + \vec{K}\vec{U} \tag{11}$$

This way coupling of both processes is obtained. The state variables for wave number vector, the wave amplitudes, the current field and the water depth became interdependent.

The equations for calculation of the wave amplitude are obtained by considering conservation of energy E. For a system of constant water depth the sum of potential and kinetic energy is given by

$$E = (1+\xi)\frac{1}{2}\rho g a^2 ; \xi = \frac{\delta}{4k}(1+G) \tag{12}$$

The second term represents diffraction and thus is an extension to the classical approach. It is obtained straight forward from the relations given above.

The energy concentrated in a cell propagates into areas of undisturbed surface. This leads to a propagation equation for the wave amplitude of the form

$$a_{,t} + \frac{1}{2a}\left(\left(U_i + C_{Ei}\right)a^2\right)_{,i} + \frac{S_{ij}}{\rho g a}U_{j,i} - \frac{U_i\left(T_i - T_i^b\right)}{\rho g a} = \frac{\epsilon}{\rho g a} \tag{13}$$

The advective transport relates to the current velocities U_i and the energy transport velocity C_{Ei}.

$$C_{Ei} = \frac{C_g}{(1+\xi)} \tag{14}$$

The deformation of propagating waves with respect to height and direction leads to velocity and pressure variations which are summed up in the radiation stresses term S_{ij}. Within the linear wave theory the following expressions are obtained from momentum conservation

$$S_{ij} = \frac{1}{16}\rho g a^2 (1+2G) \tag{15}$$

Reviewing the equation so far, they have been obtained from conservation principles.

Further coupling mechanisms are contained in empirical relationships contained in equation (13).

The friction terms T_i at the free surface is commonly represented by a quadratic relation to the wind field components.

The friction at bottom T^b_i is commonly specified in terms of current velocities, orbital water motion, water depth and roughness parameters corresponding to the sea bed grain size and shape as well as to some form parameters such as ripples, dunes etc.

The breaking process of waves is represented by ε. Different approaches are possible. DAILLY, DEAN and DALRYMPLE propose a relation based on the energy dissipation proportional to the difference between the actual and the stable wave energy flux. Other relations may be used related to wave shape parameters.

2.3 Current Model

For engineering applications the current field is commonly modeled in terms of vertically integrated fluxes resp. velocities. The corresponding differential equations are obtained from mass and momentum conservation principles. Omitting Coriolis and density variations as well as turbulence effects which can only be obtained by some parametrization, the following equations hold:

$$U_{,t} = -U_j U_{i,j} - g\,\eta_{,i} - \frac{S_{ij,i}}{\rho\,(d+\eta)} + \frac{T_i + T_i^{\,b}}{\rho\,(d+\eta)} - \frac{p_{,i}}{\rho\,(d+\eta)} \tag{16}$$

$$\eta_{,t} = -\big(U_i\,(d+\eta)\big)_{,i} \tag{17}$$

Coupling between momentum (16) and mass (17) equation in terms of η and U is evident.

The last term in (16) represents the forcing from the atmospheric pressure field and thus the coupling with the wind field equations.

The radiation stresses and friction at the sea bottom are the same as for the wave equations .

The friction at the free surface refers to the wind field components.

2.4 Transport Model

Currents and waves act on the sea bed picking up and releasing material. This material is transported in the water body as a concentration C of material. It is assumed that the concentration is equally distributed over the water depth which is in agreement with the assumption of vertically averaged flow velocities. The transport is given by

$$C_{,t} = -U_i C_{,i} + \left(D_i C_{,i}\right)_{,i} + S \tag{18}$$

This equation couples back to the current model in terms of flow velocities.

The equation contains basically two parametric quantities. The dispersion coefficient D is dependent on the turbulence characteristics of the flow field under currents and waves, the density of concentration, the bed material properties and the bed forms. The same holds for the source and sink term S representing the entrainment and release of material at the bottom.

An extension of this single equation into a set of equations each standing for a portion of the grain size distribution curve and/or suspended and bed load transport would be straight forward.

2.5 Bed Evolution Model

Depending on the source and sink term there will be areas where material settles or erodes. The amount of material entering the water body is obtained by balancing the concentration C, pick up and release of matarial over the water depth from bottom to surface

$$q_i = \int U_i C \, dx_3 + q_{bi} \tag{19}$$

Due to mass conservation this material leads to a morphodynamic evolution of the sea bed according to

$$\left(1-\kappa\right) d_{,t} = -q_{i,i} \tag{20}$$

This equation couples back to the wave and current model due to the changing water depth.

The parametric term (1-κ) represents the porosity of the bed material.

3 Numerical Scheme

The derived equations are fully coupled. They are of hyperbolic/parabolic type. This allows for application of the same numerical algorithm to all equations.

The solution is obtained by a Finite Element Petrov-Galerkin scheme. As the equations do not contain higher than second order derivatives triangular elements with linear interpolation functions may be chosen. Second order derivatives are removed by partial integration.

Arranging the equations in matrix form with $\underline{A}$, $\underline{B}$ containing the coefficients and $\underline{X}$ the state variables and introducing the interpolation function φ leads to

$$\iint \left(\underline{\varphi}+\tau\,\underline{A}(\varphi)\right)\left(\underline{X}(\varphi)_{,t}+\underline{A}(\varphi)\,\underline{X}(\varphi)+\underline{B}\right)dx_1\,dx_2=0 \qquad (21)$$

The weighting consists of two components. The first weighting step by φ corresponds to the standard Bubnov-Galerkin procedure which is well known to preserve peaks but which is highly oscillatory. The second component in the weighting procedure corresponds to the error squared method which has good smoothing properties. The second component is premultiplied by τ which represents an upwind parameter. It is determined on the element level directly from the actual propagation conditions.

The equation system obtained this way is implicit.

$$\underline{M}\,\underline{X}_{,t}=\underline{Q} \qquad (22)$$

The computational expense is reduced by splitting the matrix $\underline{M}$ into left, right and diagonal matrix contributions

$$\left(\underline{L}+\underline{D}+\underline{R}\right)\underline{X}_{,t}=\underline{Q} \qquad (23)$$

thus obtaining an explicit form

$$\underline{D}\,\underline{X}_{,t}=\underline{Q}-\left(\underline{M}+\underline{L}+\underline{R}\right)\underline{X}_{,t} \qquad (24)$$

The time integration may now be performed iteratively with standadrd schemes. The first order Euler scheme turned out to be suffiently accurate and most economic.

As all differential equations are of propagation type, the same numerical scheme is being applied to all equations. Thus a short and clearly structured coding is possible. As the solution procedure has been reduced to an explicit scheme, the computational expense is linear proportional to the number of equations and terms involved as well as to the number of elements or nodes of the discrete system.

4 Coupling of Equations

The derivation of equations, the numerical scheme and the way of coding allow for coupled as well as for decoupled calculation of different processes. Coupling depends on physical conservation laws as well as on parametric empirical relations.

For applications processes in the field may be split according to scales in time and space.

4.1 Time Scale

The simulation of short time events in the range of minutes mainly concerns the reproduction of actual physics. Due to the assumption of monochromatic waves and vertically integrated currents as well as concentrations the model will only give limited coincidence with reality of nature. Other approaches may be more appropriate for this situation.

In engineering applications either maximum/minimum conditions have to be analysed or mean conditions be determined. For this purpose the system of equations may be quite useful.

The wave field is mainly generated by wind forces. Typical variations of wind with respect to strength and direction are in the range of few hours. For deep water conditions where waves travel undisturbed radiation stresses are very small. Waves and currents may be treated independently.

As soon as the waves are deformed in near beach areas the radiation stresses generate considerable currents which deform the incoming wave field. A coupled simulation becomes necessary.

The propagation of waves is strongly dependent on the actual water depth. This may vary considerable in tidal waters or during storm events which leads to a time scale of hours and days. These variations are dominantly related to the current model. For weak wind effects or deep water the wave calculation may be omitted. In shallow water however, where breaking of waves occurs, a coupled calculation is necessary.

Water depth is also changing due to morphodynamic processes. The time scale of relevant erosion at beaches may either be in the order of hours during storm situations or weeks and months during mean conditions. A coupled simulation may well be possible for few hours of real time, for long range processes computing time will become a limiting factor.

4.2 Space Scale

The economy of calculations depends on the level of closure of models. This again is determined by the processes to be modeled.

The simulation of surface waves demands for a spatial resolution of about 1/10 the of the length of the waves. This high resolution is necessary in near beach areas with breaking waves.

For simulations in deeper water, where the radiation stresses are not that large, simplifications may be allowed. For computer runs over real time periods of hours the wave model may be calculated at high resolution only every hour and the forces interpolated in time. The underlying current model has to be run continuously at resolutions of several hundert meters which is fine enough for resolving long period seiches or tidal waves. Special software routines have to support this kind of application.

Morphodynmic processes are of main interest to the engineer. Those processes demand for high resolution in the near beach area and large spatial extension of the investigation areas to reproduce boundary conditions for sediment transport and currents correctly. The computing time becomes that high, that wave and even current processes have to be interpolated and averaged.

4.3 Parametrizations

The strong coupling of the equations is determined by the conservation laws. The empirical terms for friction, resuspension of material, breaking of wates, dispersion etc represent weak coupling only and can hardly be used for justifying coupling and decoupling of equations. Moreover in many cases there are different terms applicable - breaking description by steepness or energy transfer - which demand for quite some practical experience from application. The same holds for the choice of coefficients contained in parametric terms. As these have been obtained from field or experimental data analysis whereas the equations have been derived under assumptions for making physics accessible to mathematical description modification may become necessary.

4.4 Input Data

Models are driven by boundary conditions. These have to be specified in agreement with the simplifications made within the models.

Wind data are mostly available for few locations only. Interpolation to the model areas is hard in case of deformation of the wind field near structured and high coastlines.

Wave data have to be obtained from field measurements in which quite different processes may be superimposed. Thus the data analysis is tricky and time consuming. The accuracy remains limited. Determination of wave directions for instance is realistic only within an error of about 10 degrees.

Knowledge about currents at boundaries demand for field compaigns. As these are very expensive quite often water level measurement are taken and accepted.

Morphodynamic is very difficult to monitor. It is nearly impossible to do echosounding for large areas continuously. Generally data have to be taken which are available anyway .

Besides the scarcity of data there are two further problems. The first concerns the fact that data are hardly available synoptically. This makes coupled simulation strongly dependend on plausibility calculations about long time series of measurements and experience from the field.

The second aspect concerns the selection of input data with respect to the intended goal of a study. For calculation of waves propagating over a fixed bed the mean or significant wave may be selected from a spectral analysis of measured waves. However for calculation of sediment transport the mean wave must not necessarily the significant wave for initiating morphodynamic transport processes. Research still has to be done in this area.

5 Application

Numerical studies are currently being performed in different environments. It turns out that for areas of several 10 kilometers of extension and deep water conditions coupling is not necessary. Closer to the shoreline, where breaking processes lead to strong radiation stresses, coupled calculations become necessary. An application of this kind is given in the figure attached. The four segments are showing:

- Upper left: Depth distrubition for an idealised beach area with reafs and gaps in between, in which rip currents may be forming. In the center an artificial sandfill is modeled which reaches from the shore to the reef.
- Upper right: Wave crests and wave heights approaching the beach. The wave direction is inclined by 5 degrees to the normal to the shoreline. The prescribed wave heigth in deep water was 3 m.
- Lower left: Generated currents represented by a vector field of flow velocities. The longshore currents from top to bottom of the sketch as well as the seaward rip currents in the gaps between the reefbodies are clearly seen. The figure represents a stationary state in the initial phase of bathymetry.
- Lower right: Zones of erosion and sedimentation after a calculation for 5 days.

The calculation have been performed on a grid of about 20000 nodes with a resolution of about 5 meters in the near beach area. The wave set-up at the beach was in range of 0.2 m which agreed well with observations. The longshore currents were in the range of observations too.

6 Conclusion

A closed set of differential equations has been derived for monochromatic waves on sloped bottom, vertically integrated currents, sediment transport and morphodynamic changes of the sea bed. The set of equations is of propagation type. This allows for application of the same numerical solution scheme, which is based on linear Finite Elements and on an upwinding Petrov-Galerkin procedure, to all equations simultaneously. The full set of equations may be solved coupled as well as decoupled. Numerical studies are performed on different scales in time and space. Strong coupling of the equations becomes necessary in areas of strong radiation stresses. Because of computational expense calculation of different processes on different grids may be considered in future.

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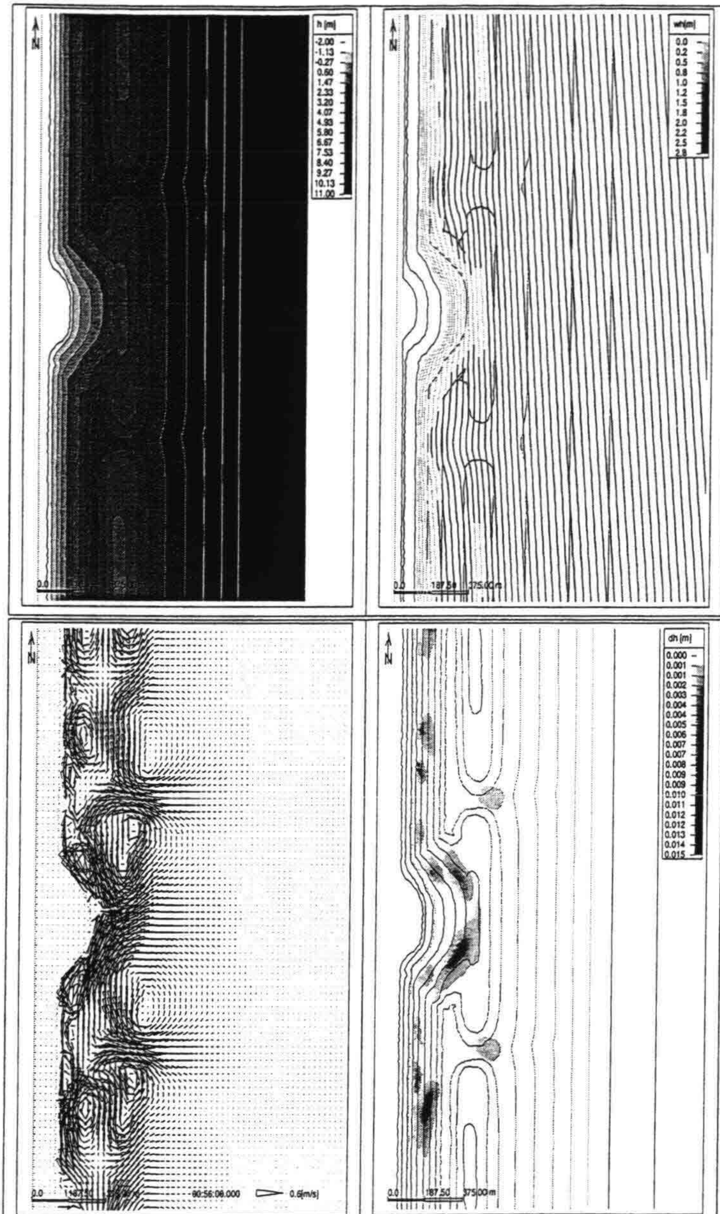

Figure 1: Computational Results

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A Second-Order Solution of the Flap Wavemaker Problem

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Abstract

A second-order solution of the flap wavemaker problem is presented in this paper. The present solution is derived based on complex variables, which is different from previous solutions derived using real variables by Hudspeth and Sulisz(1991), and Sulisz and Hudspeth(1993b). The perturbation method is used to solve the nonlinear boundary value problem. In the second-order solution, unwanted real parts produced by multiplication of imaginary parts of the linear complex solutions are subtracted from the formulation so that correct solutions can be obtained. Complete solutions up to the second order are presented. The present analytic solutions are compared favorably with experimental results by Wu(1987). Using the present theory, the characteristics of the second-order waves generated by the flap wavemaker are studied. The waves generated in the wave channel consist of the Stokes waves and the wavemaker free waves. Analysis of the mass transport shows that the free waves generate negative value of the Stokes drift. Therefore, wavemaker generated waves satisfy conservation of mass, and have zero mass transport.

1 Introduction

The generation of waves in the wave channel provides useful information such as wave deformation, wave characteristics et al., for ocean waves in the coastal region, and therefore is very important in ocean engineering (Madsen, 1971; Flick and Guza, 1980; Wu, 1987). The wavemaker theory provides a useful tool to simulate waves in the wavemaker channel. The difficulties of analyzing the wavemaker problem are mainly on the moving boundaries at the free surface and the wavemaker boundary, and the inherent kinetic and dynamic non-linearity of the problem (Kim, 1984). Not until recently, a complete second-order solution for a generic-type wavemaker problem was proposed by Sulisz and Hudspeth(1993a). The perturbation method was used to analyze the problem, and Taylor series expansions was used for the moving free surface and the wavemaker boundaries. Real variables were used in deriving the second-order solution. The Stokes drift in the two-dimensional wave flume was calculated. The solution method was extended to calculate wave loading on the wavemaking structures (Sulisz and Hudspeth, 1993b), on a horizontal rectangular cylinder (Sulisz and Johnasson, 1992), and on submerged structures (Sulisz, 1993). An attempt of using complex variables for the second-order solution for the wavemaker problem was proposed by Lee, Liu, and Lan(1994). The method similar to Vantorre (1986) was used to obtain correct the second-order solutions.

2 Theory and solutions

2.1 Problem description

Consider a flap wavemaker located at one end of a semi-infinite channel of constant water depth as shown in Figure 1. A Cartesian coordinate system is used with origin located at still water level, x-axis pointed to the right, and z-axis pointed upward. The water depth is h and the free surface elevation η. The wavemaker is given a periodic motion with frequency σ, and the displacement function $\xi(z,t)$ defined by

$$\xi(z,t)=\begin{cases} i\dfrac{s_0}{2(h-b)}(z+h-b)e^{-i\sigma t} & , -(h-b)<z<0 \\ 0 & , -h<z<-(h-b) \end{cases} \tag{1}$$

where s_0 is the stroke, b is the height of the hinge above the bottom. The generated waves can be described by potential function $\Phi(x,z,t)$ defined by

$$\vec{V}=-\nabla\Phi \tag{2}$$

where $\vec{V}$ is the fluid velocity, ∇ fluid is the gradient operator. The boundary value problem solving for wave potential Φ can be written as

$$\nabla^2\Phi=0 \tag{3}$$

$$-\Phi_{,z}=0 \quad \text{on } z=-h \tag{4}$$

$$-\Phi_{,t}+\frac{1}{2}\left|\vec{V}\right|^2+g\eta+B(t)=0 \qquad \text{on } z=\eta(x,t) \tag{5}$$

$$-\Phi_{,z}=\eta_{,t}-\Phi_{,x}\eta_{,x} \qquad \text{on } z=\eta(x,t) \tag{6}$$

$$-\Phi_{,x}=\xi_{,t}-\xi_{,z}\Phi_{,z} \qquad \text{on } x=\xi(z,t) \tag{7}$$

in which g is gravitational constant, $B(t)$ is Bernoulli constant, the subscript comma indicates differentiation. Equation (7) is the kinematic condition of the wavemaker boundary. The dynamic condition Equations (5) and kinematic condition Equation (6) can be combined as

$$-\Phi_{,tt}-g\Phi_{,z}+(\vec{V}^2)_{,t}+\vec{V}\cdot\nabla(\frac{1}{2}\vec{V}^2)+B'(t)=0 \tag{8}$$

At infinite x the waves propagating outward satisfy radiation condition, and are of finite value. The boundary value problem is nonlinear, and is defined on moving free surface boundary and wavemaker boundary. The problem can be approximated by Taylor series expansion about equilibrium positions $z=0$ and $x=0$. The perturbation method is then used to obtain boundary value problems for each orders.

2.2 The first order problem

The first order problem can be expressed as

$$\nabla^2\Phi_1 = 0 \qquad , 0 \le x < \infty \ ; \ -h \le z \le 0 \tag{9}$$

$$-\Phi_{1,z} = 0 \qquad , z = -h \tag{10}$$

$$\Phi_{1,tt} + g\Phi_{1,z} = \frac{dB_1}{dt} \qquad , z = 0 \tag{11}$$

$$\Phi_{1,x} \doteq -\xi_{,t} \qquad , x = 0 \tag{12}$$

And the free surface elevation of the first order η_1 be expressed by

$$\eta_1 = (1/g)(\Phi_{1,t} + B_1) \qquad , z = 0 \tag{13}$$

Since the first order solution is periodic in time, it is obvious that the Bernoulli constant B_1 in Equation (13) is zero. The first order solution is well known and can be summarized as following,

$$\Phi_1 = i\frac{T_0}{K_1}\cosh[K_1(z+h)]\, e^{i(K_1x-\sigma t)} + \sum_{n=1}^{\infty}\frac{T_n}{k_{1n}}\cos[k_{1n}(z+h)]\, e^{-k_{1n}x}e^{-i\sigma t} \tag{14}$$

$$\eta_1 = \frac{\sigma}{g}\left[\frac{T_0}{K_1}\cosh(K_1h)\, e^{i(K_1x-\sigma t)} - i\sum_{n=1}\frac{T_n}{k_{1n}}\cos(k_{1n}h)\, e^{-k_{1n}x}e^{-i\sigma t}\right] \tag{15}$$

where

$$T_n = \frac{2\sigma s_0}{k_{1n}h}\left[\frac{k_{1n}h\sin k_{1n}h + \dfrac{h}{h-b}(\cos k_{1n}h - \cos k_{1n}b)}{2k_{1n}h + \sin 2k_{1n}h}\right] \tag{16}$$

The wave number k_{1n} is calculated from the dispersion equation,

$$\sigma^2 = -gk_{1n}\tan k_{1n}h \qquad , n = 0,1,2,\cdots \tag{17}$$

with the definition $k_{10} = -iK_1$. The gain function of the first order solution, that is the ratio of the generated wave height to the wavemaker stroke can be written as

$$G = \frac{2a_{1p}}{s_0} = \frac{2\sigma T_0}{s_0 g K_1}\cosh(K_1h) \tag{18}$$

2.3 The second order problem

The second order problem can be written as

$$\nabla^2\Phi_2 = 0 \qquad , 0 \le x < \infty \ ; \ -h \le z \le 0 \tag{19}$$

$$-\Phi_{2,z} = 0 \qquad , z = -h \tag{20}$$

$$\Phi_{2,tt} + g\Phi_{2,z} = 2\Phi_{1,x}\Phi_{1,xt} + 2\Phi_{1,z}\Phi_{z,t} - \eta_1\Phi_{1,ttz} - g\eta_1\Phi_{1,zz} + dB_2/dt \qquad , z = 0 \tag{21}$$

$$-\Phi_{2,x} = \xi\Phi_{1,xx} - \xi_{,z}\Phi_{1,z} \qquad , x=0 \tag{22}$$

The free surface elevation of the second order η_2 has the following form,

$$\eta_2 = \frac{1}{g}\left\{\Phi_{2,t} - \frac{1}{2}\left(\Phi_{1,x}^2 + \Phi_{1,z}^2\right) + \eta_1\Phi_{1,tz} + B_2\right\} \qquad , z=0 \tag{23}$$

Since the second order problem contains two non-homogeneous boundaries, therefore, in order for the problem to be solvable it is first divided into two parts,

$$\Phi_2 = \Phi_2^s + \Phi_2^f \tag{24}$$

where Φ_2^s is to satisfy the problem with non-homogeneous free surface boundary condition, Φ_2^f satisfies the problem with non-homogeneous wavemaker boundary condition. And correspondingly free surface elevation be expressed as

$$\eta_2^s = \frac{1}{g}\left\{\Phi_{2,t}^s - \tfrac{1}{2}\left(\Phi_{1,x}^2 + \Phi_{1,z}^2\right) + \eta_1\Phi_{1,tz} - B_2\right\} \tag{25}$$

$$\eta_2^f = \frac{1}{g}\Phi_{2,t}^f \tag{26}$$

Since complex variables are used in the derivation, multiplication of the first order solution can produce unwanted real parts and need to be discarded. The method used by Vantorre (1986) to avoid this problem can be written as

$$\mathrm{Re}\left(\alpha e^{-i\sigma t}\right)\cdot \mathrm{Re}\left(\beta e^{-i\sigma t}\right) = \mathrm{Re}\left[\tfrac{1}{2}\left(\alpha\cdot\beta e^{-i2\sigma t}\right) + \tfrac{1}{2}\alpha\cdot\overline{\beta}\right] \tag{27}$$

where α and β are complex, and the over bar indicates complex conjugate. Equation (34) means real parts of the two sides of the equation are equal. Note that from Equation (27) a time-dependent term and a time-independent term are produced, and similarly results in Equations (21)-(23). By applying wave periodicity the second order Bernoulli constant B_2 in Equation (23) can be shown to be $-T_0^2/4$. The solution of Φ_2^s and Φ_2^f take the forms of $\Phi_2^{*s} + \varphi_2^s$ and $\Phi_2^{*f} + \varphi_2^f$ where Φ_2^{*s} and Φ_2^{*f} are time-dependent functions, and φ_2^s and φ_2^f and are time-independent. The solutions can be summarized as followings.

$$\Phi_2^{*s} = \sum_{n=0}^{\infty}\sum_{m=0}^{\infty} C_{2nm}^s \cos\left[(k_{1n}+k_{1m})(z+h)\right]e^{-(k_{1n}+k_{1m})x}\, e^{-i2\sigma t} \tag{28}$$

$$\varphi_2^s = \sum_{n=1}^{\infty} D_{2n0}^s \cos\left[(k_{10}-k_{1n})(z+h)\right]e^{(k_{10}-k_{1n})x}$$

$$+ \sum_{n=0}^{\infty}\sum_{m=1}^{\infty} D_{2nm}^s \cos\left[(k_{1n}+k_{1m})(z+h)\right]e^{-(k_{1n}+k_{1m})x} \tag{29}$$

where the coefficients C_{2nm}^s, D_{2n0}^s and D_{2nm}^s are integration coefficients. The solution of Φ_2^{*f} can be written as

$$\Phi_2^{*f} = \sum_{\ell=0} C_{2\ell}^f \cos\left[k_{2\ell}(z+h)\right]e^{-k_{2\ell}x}e^{-i2\sigma t} \tag{30}$$

where $k_{20} = -iK_2$, $k_{2\ell}$ satisfies the second order dispersion equation

$$4\sigma^2 = -gk_{2\ell} \tan k_{2\ell}h \quad ,\ell = 0,1,2,3,\cdots \tag{31}$$

The coefficient $C_{2\ell}^f$ in Equation (30) can be obtained by utilizing the orthogonal function. The solution of φ_2^f can be written as

$$\varphi_2^f = D_{20}^f x + \sum_{\ell=1}^{\infty} D_{2\ell}^f \cos\left[\mu_\ell (z+h)\right] e^{-\mu_\ell x} \tag{32}$$

where D_{20}^f and $D_{2\ell}^f$ are integration coefficients.

$$\mu_\ell = \frac{\ell \pi}{h}, \quad \ell = 1,2,3,\cdots \tag{33}$$

2.4 The free surface elevation

The free surface elevation η calculated from wave potential contains the first order η_1 and the second order η_2. The second order quantity equals $\eta_2^s + \eta_2^f$. η_2^s is induced from the free surface boundary condition,

$$\eta_2^s = \eta_2^{*s} + \overline{\eta}_2^s \tag{34}$$

where

$$\eta_2^{*s} = \sum_{n=0}^{\infty}\sum_{m=0}^{\infty} A_{2nm}^s e^{-(k_{1n}+k_{1m})x} e^{-i2\sigma t} \tag{35}$$

$$\overline{\eta}_2^s = \sum_{n=1}^{\infty} \overline{A}_{20n}^s e^{(k_{10}-k_{1n})x} + \sum_{n=0}^{\infty}\sum_{m=1}^{\infty} \overline{A}_{2nm}^s e^{(k_{1n}+k_{1m})x} \tag{36}$$

Equation (34) includes a time-dependent term and a time-independent term. The time-dependent term can be further grouped into a pure propagation component η_2^{*spp}, an evanescent-propagation multiplication component η_2^{*spe}, and a pure evanescent component η_2^{*see}. η_2^f is induced by the wavemaker condition

$$\eta_2^f = \sum_{n=0} a_{2n}^f e^{-k_{2n}x} e^{-i2\sigma t} \tag{37}$$

where

$$a_{2n}^f = -\frac{i2\sigma}{g} C_{2n}^f \cos k_{2n}h \tag{38}$$

Equation (37) includes propagation wave η_2^{fp} ($n=0$)and evanescent waves η_2^{fe} ($n \geq 1$). From Equations (15), (35), and (37) the propagation waves away from the wavemaker can be written as

$$\eta^p = a_{1p}\cos(K_1 x - \sigma t) + a_{2p}^{*s}\cos[2(K_1 x - \sigma t)] + \left|a_{2p}^f\right|\cos(K_2 x - 2\sigma t + \theta_2) \tag{39}$$

where the phase angle θ_2 is defined as

$$\theta = \tan\left[\mathrm{Im}\left(a_{2p}^f\right)/\mathrm{Re}\left(a_{2p}^f\right)\right] \tag{40}$$

2.5 The mass transport

The mass momentum flux or mass transport can be expressed as

$$M = \left\langle \int_{-h}^{\eta} \rho U dz \right\rangle \tag{41}$$

where U is the horizontal fluid velocity, $\langle\cdot\rangle$ indicates time average of wave period. Equation (41) is calculated by first applying Taylor series expansion about z=0, then substituted into velocities up to the second order. Equation (41) can be rewritten as

$$M = M_{\Phi} + M_{\varphi} \tag{42}$$

where M_{Φ} represents result of time-dependent functions, M_{φ} represents result of time-independent functions. The time-dependent result can be further formulated as

$$M_{\Phi} = M_{\Phi}^{e}(x) + M_{\Phi}^{\infty} \tag{43}$$

where

$$M_{\Phi}^{e}(x) = \rho \frac{a_{1p}}{2}\left[\sum_{n=1}^{\infty} \frac{T_n}{k_{1n}} \cos(k_{1n}h) e^{-k_{1n}x}\left[k_{1n}\cos(K_1 x) - K_1 \sin(K_1 x)\right]\right] \tag{44a}$$

$$M_{\Phi}^{\infty} = \frac{\rho g K_1 a_{1p}^2}{2\sigma} \tag{44b}$$

Equation (44b) is the so called Stokes drift. On the other hand, the time-independent result in Equation (42) can be expressed as

$$M_{\varphi} = M_{\varphi}^{e}(x) + M_{\varphi}^{\infty} \tag{45}$$

where

$$M_{\varphi}^{e}(x) = -\rho\left(\frac{a_{1p}}{2}\right) \sum_{n=1}^{\infty} \frac{T_n}{k_{1n}} \cos(k_{1n}h) e^{-k_{1n}x}\left[k_{1n}\cos(K_1 x) - K_1 \sin(K_1 x)\right] \tag{46a}$$

$$M_{\varphi}^{\infty} = -\rho D_{20}^{f} h = -\frac{\rho g K_1 a_{1p}^2}{2\sigma} \tag{46b}$$

Equation (46b) indicates that there is return flow toward the wavemaker. Notice that in the wavemaker theory the Stokes drift M_{Φ}^{∞} is balanced by the mean return flow M_{φ}^{∞}. This is true because in the wavemaker problem the Laplace equation is solved which indicates the conservation of mass.

3. Results and Discussion

To show accuracy of the present theory, analytic solution up to the second order is compared to experimental results by Wu (1987). The hinged height of the wavemaker is 0.035m, the period and stroke of the wavemaker motion are 1.23sec and 0.12m, respectively. The constant water depth is 0.4m. The data acquisition sampling rate was 74 times per second. In Figure 2 the real line is analytic solution and circles represent experimental results, Figure 2(a) is water elevation at four times water depth, and Figure 2(b) at five times water depth. It can be seen that the comparisons show very good agreement. Using the present analytic solution, wave forms generated by the wavemaker under different conditions are studied. As shown in Figure 3 non-dimensional wave height versus distance from the wavemaker, Figure 3(a), 3(b), and 3(c) are for $K_1 h$=0.5, 1.5, and 3.0, respectively. Sinusoidal wave forms are generated for relative shallow depth, whereas the deeper depth increases the irregular wave forms. In figure 3(c) second wave peaks are shown in the wave troughs.

The analysis of the wavemaker theory shows that the second order component resulted from the second order non-homogeneous free surface condition and the second order wavemaker boundary condition. The free surface condition induced the well known Stokes second order wave, while the wavemaker condition generated free waves. The analysis of mass transport showed that the Stokes wave induced Stokes drift, and the free wave generated an amount negative to the Stokes drift. In other words, in the wave channel the generated waves carry zero mass transport, that is, the fluid mass remains conserved. One should be aware that in this analysis a semi-infinite channel is considered, therefore, no effects of end wall reflection induced.

4. Conclusion

In this paper detailed derivation of the second order wavemaker theory using complex variables is presented. The flap type wavemaker is considered. The analytic solution is verified by the solution derived using real variables. Comparison with experimental results also shows favorable agreement. From the analysis the second order wavemaker solution consists of the Stokes waves and the wavemaker free waves. The analysis of mass transport also indicates that the former induces Stokes drift, and the later generated an equal but negative amount. Therefore, in the wave channel the generated waves up to the second order carry no mass transport, which implies the solution satisfies mass conservation.

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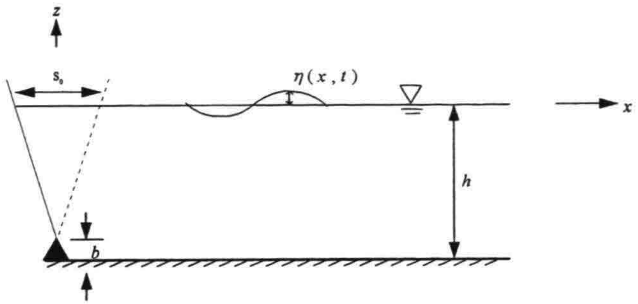
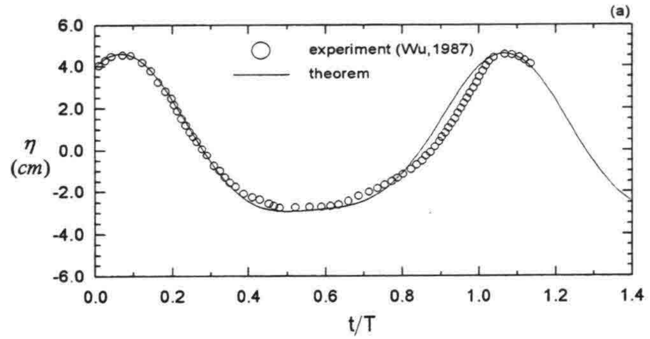


Figure 1 Definition sketch of the flap wavemaker problem.



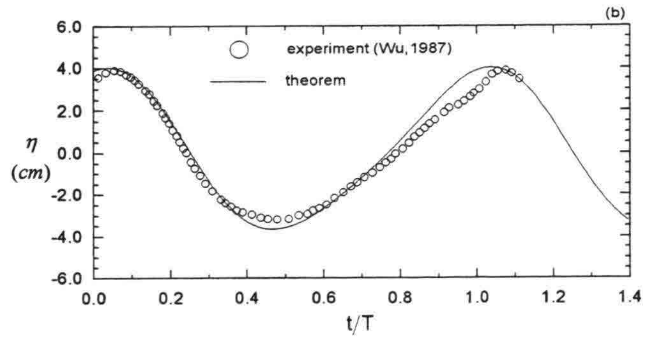
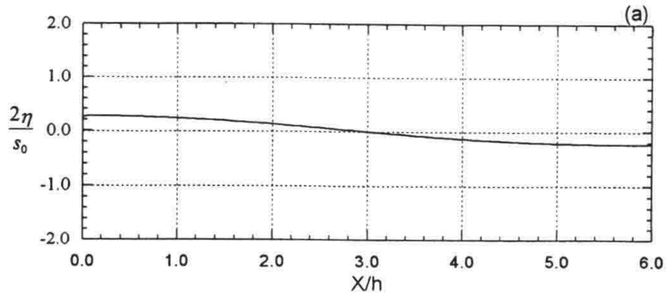
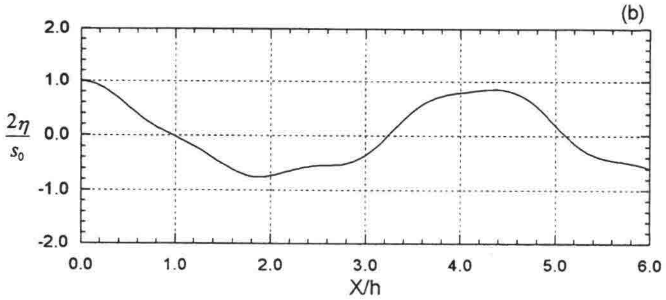


Figure 2 Comparison of present analytic solution with experiment by Wu(1987). T=1.23sec, s_0=0.12m, h=0.4m. (a)x/h=4 (b)x/h=5





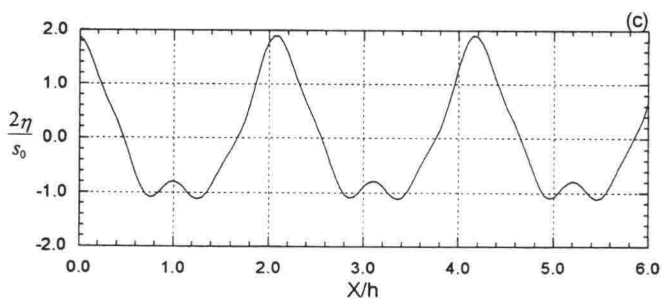


Figure 3 Wave forms generated by the wavemaker s_0/h=0.3, b/h=0. (a) K_1h=0.5, (b) K_1h=1.5, (c) K_1h=3.0.

ACTIVITIES OF COASTAL DEVELOPMENT IN TAIWAN

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ABSTRACT

Taiwan is an inland, with the promotion of the national fast-growing economic Development, Taiwan is no doubt to the advanced on a par within four Asia Dragons.

Owing to the industrious and hard working people as well as correct and sound national development policy. In view of limitation of restricted land of the inland, it is total to the medium and small scale enterprises to establish overseas offices or factories in Mainland China, which draw particular attention of the government to regulate and supply toward releasing more farm land and reclaiming more space from the shoreline aggressively. This article briefly reviews the existing land-use attics of Taiwan coastline and the on-going islandwise land-reclaiming activities as well as some perceptive of long-term developments.

1. INTRODUCTION

Taiwan is situated in the Pacific Ocean about 160 Kilometers (100 miles) from the south-eastern coast of China main-land. Located about midway between Korea and Japan to the North and Hong-Kong and Philippines to the South. Shaped roughly like a tobacco leaf, Taiwan is the place of 36,000k ㎡ land with 1,100km long coastline and population exceed 21 millions which makes the island of the world's second densely-populated area. Owing to the recent 30 years dynamic development of industry, most of the land resources had been put into exploiting. Following to the government releasing some farmland, mountain slope, it is apparent the shortage of lands. still remains to the unsettled due to high-technological development, which enhances or stimulates the set-up of more new industry zone in sense of going along with more land to be required and force the government to work out new land reclamation policy, regardless of the fact that 5,000 hectares had already been producing in hand. Furthermore with the rapid increasing of economic/social development and making full use of the latest achievements in technology and maritime resources, the more opportunity is giving for re-development of the water front area with a variety of recreation or amenity use as well as industry zoning instead of some original fishing or farming purpose.

As shown in the followings are the brief descriptions of existing shoreline development, policy formulation, planning and future perspective etc.

2. Coastline Environments of Taiwan

2.1 The category and features of coastline

In most cased of western island, it is characterized mainly by the full range of the north and south end corner, however rocky features displays in major regions of northeast, southeast and east coast around Taiwan. The gravity of land reclamation activity lies in the west of Taiwan see Fig.1 (The category and Features of Taiwan coastline).

2.2 The oceanographic and meteorologic environmentals.

Taiwan's climate is of subtropical /tropical, oceanic regions to be subject to topographic features and monsoons, which yields a rather high-temperature, rainy and windy with average annual 2,500mm rainfall. There are 129 rivers or creeks passing through whole islands, with strong erosion action inevitably caused substantial distribution due to steep mountain slopes and high flow speed. The rainfall concentrates in raining season and distributes unevenly generating an unbalance in between demand and supply of stable water resources.

Regarding to the oceanographrical aspects, it is apparent that the six months monsoon ,from Nov. to the following Apr.,and typhoons of summer(usually 2 ~ 3 times) are drawn to the main provisions of calibrating the forecast of wave current or other factors.

In west-coast, it is usually anticipated 4 ~ 5 meters maximum wave height(HO) of monsoon wave with 8 ~ 10seconds period (TO) and 9 meters wave height with 13 seconds period of Typhoon wave, however in east-coast, it is expected 5 meters wave height with 9 ~ 11 seconds period of winter monsoon and 16m wave height of Typhoon as well as considerable times of 10 seconds period of long wave (swell) . As to the tide, the coast of central region of west island (Hsin-chu. Taichung and Chang-Hua) displays the highest 4.6 meters tide-range and only 1 Meter in the North and South end of the island.

2.3 The social and economy features

At the and of 1994, Taiwan's population exceeded 21 million which makes the island population density 587 per square kilometer ranking 46 of global population.

The industrial structure makes a show of service trades (i.e. the third industrial circles)to the progressively approaching to dominate the Taiwan economic activities.

The gross net production (GNP) achieves more than $10,000 US dollars several years ago.

3. Policy of National Land Development

3.1 The objectives

In the interest of environment conservation and permanently development, the policy is established so as to enhance the reasonable utilization of land in a practical manner to promote the living standard of general public and take Account of the requirement of industrial environments that includes:

- The protection of national biological environment to ensure perpetual development of national resources.

- The promotion of industrial development environment to sereure high-quality of living standard in phase of construction activities.
- The improvement of human living environment to organize the construction of Taiwan in maintaining a democratic economy system of development.

3.2 The principles of planning

- To respect free market mechanism in a way to set-up an efficient development faculty.
- To fulfill the perpetual development philosophy.
- To ensure equitable distribution of national land development.
- To make sure the corporation of local government and private enterprise in course of national land development.

3.3 The strategies of land development

- To alter land use purpose of farming area and release the relatively low-grade Farm land.
- To divide national land into restricted development region and unrestricted region respectively.
- To manage the construction of Taiwan to be a hub for business operations in Asia-Pacific (or Asia-Pacific Regional Operation Center).
- To adjust the industrial structure and regional nature.
- To ensure life circle construction schemes so as to curtail the distinction of city and rural area.

4. The Existing Statues of Coastline Development

4.1 Activities of land development

Some developed and under-developing projects are named as shown in Table 1.

4.2 Engineering consultants

The major engineering consultants involving into the land development are :

- Sino-tech.
- CECI (China Engineering Consultants , Inc.)
- Others

4.3 Construction

In past years, only the governmental agency be taking the development mission as :

- RSEA (Ret-Ser Engineering Agency) , and
- BES Engineering Corporation

There are some private sectors investing the land development as :

- Formosa Plastic Company, and
- Steel Mill

5. Future Prospects

The future prospects are liable to rational solution to coastline development so as to achieve the aims of long-range prospective as following:

- To develop a perpetual management and even out biological environment.
- To promote a high-quality of living environment.
- To approach a high-efficiency production environment.
- To create a intense but orderly national space structure.

In the past, the most of coastline lands are mainly hinged around the purpose of commercial part, fisher village, aquatic farm and salt pan use. However, owing to the fast-growth of population, urbanization expansion and rapidly economic development which enhance a large scale demand of acquiring more land, it is envisaged that some low-graded shore lands will be put in hot pursuit of competing focus by land developer. For the time being, the coast on-shore lands are generally facilitated for the following 19 purposes:

- Agriculture
- Stock raising
- forestry
- Salt pan
- Aquatic farm
- Mining or quarry
- Sightseeing spots or recreation
- Ports (commercial or fishing)
- Residential or village
- Grave yard
- Industry area
- Power plant
- Air port
- Shore land, inner land transportation facilities
- Sewerage treatment or garbage disposal
- Shore-protection
- Eco-environment protection

- Drainage channel or tide-prevention
- Oil or gas extraction

Up to now, the west coastline developments lay particular stress on industry inducing a large scale of industrial belt, which may create the loss of shoreline and bear great impacts upon environment alteration as well as come into conflict with ecological / social system. In view of the above-mentioned future prospective aims, it is therefore envisaged that the coastline development will bring toward to maintain the environment protection as well as recreation purpose. Secondly, in early stage, it is in fact that the coastline developments are limited to some small-scaled area due to limitation of technology or facilities, then the developments grow rapidly and achieve several thousands' hectares by means of advanced equipment or knowledge, which already caused grate concern of environmental impacts. In reviewing the past and look forward to the future, it is anticipate that the coastline development of Taiwan might eventually target to some medium-scaled size of land requirement in a way to minimize the adverse impacts upon the environment.

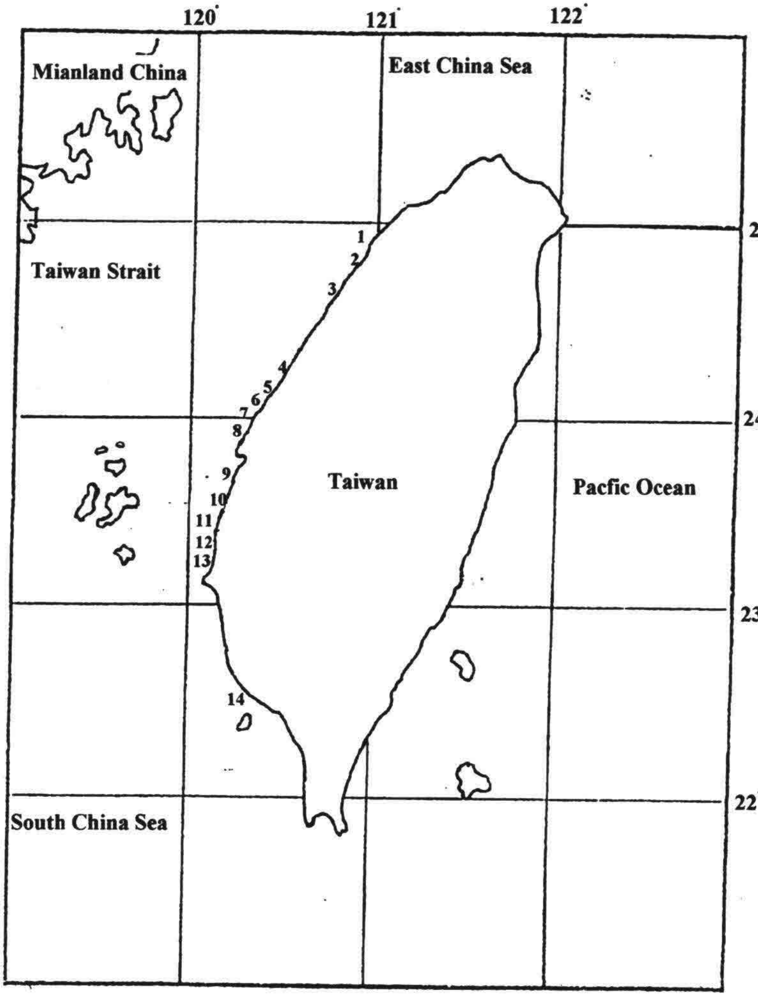
6. Summary

In brief, with the fast-growth of population and economy development of Taiwan, it is apparent that the merit of efficient utilization of limited shore land will be the most pressing task at the moment. In view of the lack of integrated and skilled management of the past coastline development and insufficiency of trains manpower and equipment, I recommend the policy of promotion of coastline development shall be the matter of great urgency and concentration placed on the following:

- To coordinate national land development plan and set up or regulate the related policy,
- To establish basically localized planning strategy.
- To educate/train the specialized professionals.
- To cooperate with foreign agency.
- To generate the introduction of advanced construction technology or equipment.
- The principle of land-releasing policy shall be based upon " Even share profits and equal access to interests ".

7. References

Hsu, Y.S.	Planning Of Tidal Land Development,1980
CUTA	Proceedings Of Land Development,1994
DB/MOEA	Industrial Districts In Taiwan, 1995
CEPP	National Land Development Plan,1996
CUTA	Study For Under-Sea Construction And Control, 1997



1.Hsin-Chu Pilot	6.Chang-Hua Amusement park	11.Chia-Yi
2.Shaing-Shan/ Hsin-Chu	7.Fang-Yan/ Chang-Hua	12.Pu-Dai/Chia-Yi
3.Tung-Shao/ Miaoli	8.Ta-Chang/ Chang-Hua	13.Yen-Tan/ ChiaYi
4.Tai-Chung-Harbor	9.Yun-lin Offshore industrial	14.South-Star Project
5.Chang-Hua Industrial	10.Ao-Ku	

Fig .1 Land Reclamations in Taiwan

Table 1 Land Reclamation along West-Coast of Taiwan

No.	project	Area （ha）	Developed by	Developed	underDeveloping	planing
1	Hsin-Chu Pilot	309	RSEA	◎		
2	Shaing-Shan/Hsin-Chu	1025	Taiwan Provincial Government			◎
3	Tung-Shao/Miaoli	132	Taiwan Provincial Government			◎
4	Tai-Chung Harbor	3973	Tai-Chung Harbor Bureau	◎		
5	Chang-Hua Industrial	3643	IDB/MOEA		◎	
6	Chang-Hua Amusement Park	2844	Taiwan Provincial Government			◎
7	Fang-Yan/Chang-Hua	1013	Taiwan Provincial Government			◎
8	Ta-Chang/Chang-Hua	804	Taiwan Provincial Government			◎
9	Yun-lin Offshore Industrial	15680	IDB/MOEA		◎	
10	Ao-Ku	996	Taiwan Sugar Company	◎		
11	Chia-Yi	6408	Taiwan Provincial Government		◎	
12	Pu-Dai/Chia-Yi	42	Taiwan Provincial Government			◎
13	Yen-Tan/Chia-Yi	2650	Taiwan Salt Factory		◎	
14	South-Star Project	400	Kao-Hsung City Government		◎	

Topic IV

Harbours and Coastal Structures

Elements of Design

Chairman: Ole Burkhardt

Some aspects of the use of geotextiles in the field of coastal engineering

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Abstract

Traditionally, the various functions of geotextiles and geotextile related products, which have to be considered individually or in combination are named filtering, drainage, separation, protection and reinforcement.

In the field of coastal engineering the most important function of a geotextile is to serve as a filter where filtering may be defined as a separation of mineral layers of varying grain size distribution in the presence of water. In the engineering practice the use of geotextile containers is increasingly noticed. Also new products combining the advantages of geosynthetics and mineral components are coming into the market.

Since a geotextile is a product being manufactured under controlled conditions, its application seems to be comparatively simple even if a geotextile may be an essential part of a coastal structure. However, various aspects must be taken into consideration covering the requirements during construction and installation as well as the requirements during the lifetime of a coastal structure. Severe conditions of a rough sea must be taken into account.

Aspects of applicability of geotextiles, demands and limitations are illustrated using recent examples from the German coastal areas.

1 Introduction

More than other disciplines of civil engineering coastal engineering is characterized by experience if we consider engineering practice. Dike construction can be taken as a typical example. Since more than 1000 years dikes were constructed at the German and Dutch North Sea coastline but, scientific approach to basic problems which are interrelated with design and construction, e.g. wave impact or wave run-up and over-topping, started some decades ago only. By all means research on these topics is not completed up to now.

A similar situation can be seen in the field of coastal protection, e.g. the construction of a revetment or a groin. Coastal structures were built according to many years of tradition where regional differences can be observed (5). There was no reason to change the almost empirical design criteria in case of stability of the structure and fulfilment of functional requirements. In case of a failure, however, constructions were repaired or, in case of a reconstruction, the structures were dimensioned somewhat stronger. A typical example is given by Fig. 1.

The complex interrelations existing between functional criteria of design and the construction itself were often ignored, and even today these interactions are rarely taken into account. If we consider downdrift erosion as a result of structural influences on a sandy beach, we can often observe, that new protection works are executed in the area of erosion. In my opinion, considering the sediment budget of a sandy beach, a demolition could often be the best technical solution. Such a solution is, however, being adopted very exceptionally only. There are various reasons for this fact.

One reason could be the lack of knowledge about the interferences between hydrodynamic influences, the coastal structure and an erosive sea bed. Fatal consequences not only for the adjacent stretches of a beach but also for the stability of the structure itself can be noted (comp.(12)), the latter often being caused by a failure of filter layers or scouring at the toe of a structure, where destruction often starts (Fig. 2).

Hydrodynamic loads caused by waves and currents and the morphological changes being induced by coastal structures cannot be calculated sufficiently safe for engineering purposes. Despite a remarkable progress in the field of numerical models, any prognosis remains uncertain. As a consequence, planning of coastal structures should include the possibility of their removal. Sometimes it would be really necessary to remove an existing structure, even if a demolition might be expensive. This is rather in contrast to conventional ideas of a civil engineer who is traditionally educated predominantly in the field of structural mechanics and related subjects and who consequently thinks in terms of stability only.

It is obvious that comparatively simple technical solutions based on geotextile bags or geotextile containers show a lot of advantages in this respect compared with conventional building materials such as steel or concrete. Geotextiles are economy-priced materials, and they are manufactured with controlled hydraulic and mechanical properties.

Advantages can often be noted also considering transport and installation. In some cases even a technical solution is possible only by the use of geotextile components (18, 22).

According to long-term and short-term fluctuations of the sea bed we have to look for flexible constructions, e.g. when designing the toe of a revetment.

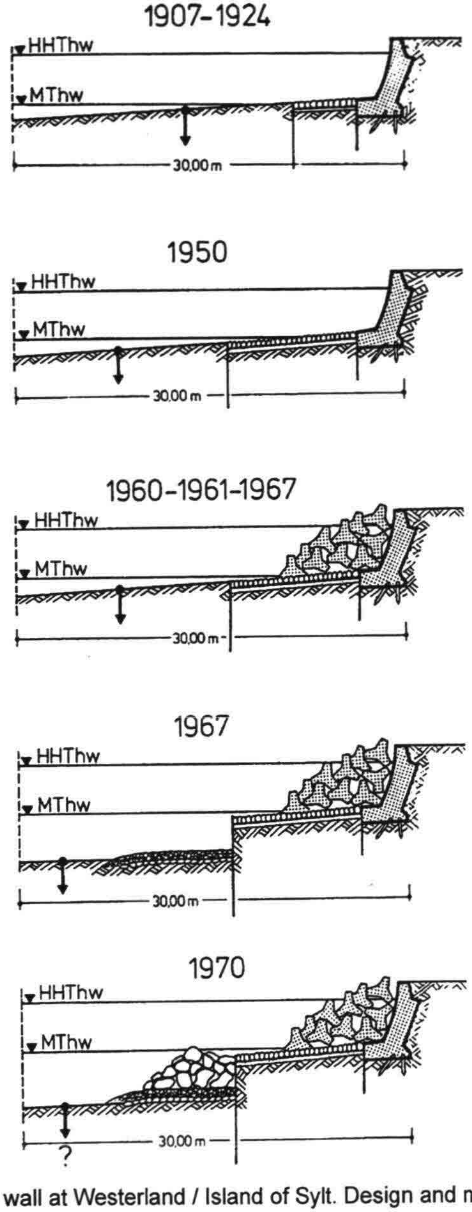


Fig. 1: Sea wall at Westerland / Island of Sylt. Design and modifications (from (1))

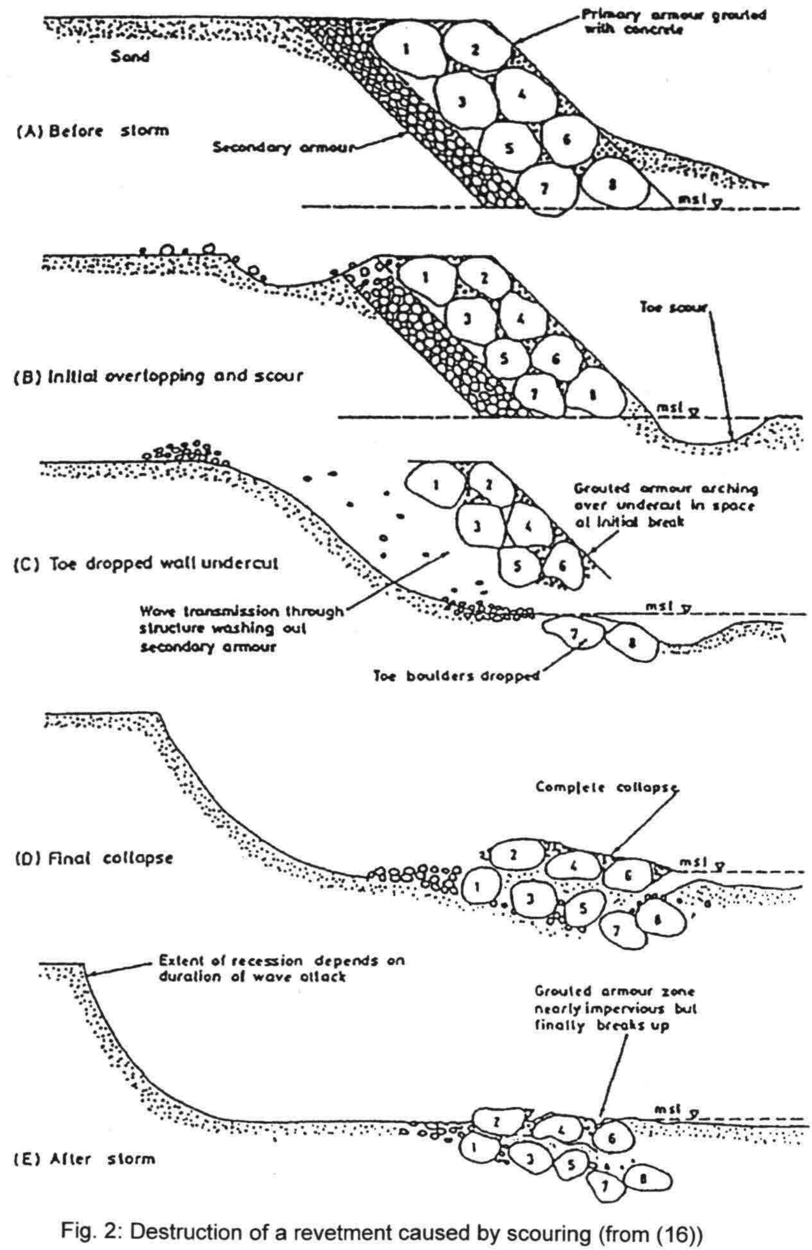


Fig. 2: Destruction of a revetment caused by scouring (from (16))

2 General remarks on the use of geosynthetics in the field of coastal engineering with respect to various functions

Functions attributed to geosynthetics as components of a coastal structure are usually as follows (4, 10, 22, 23):

Filtering, drainage, separation, protection, reinforcement, encapsulation (Fig. 3):

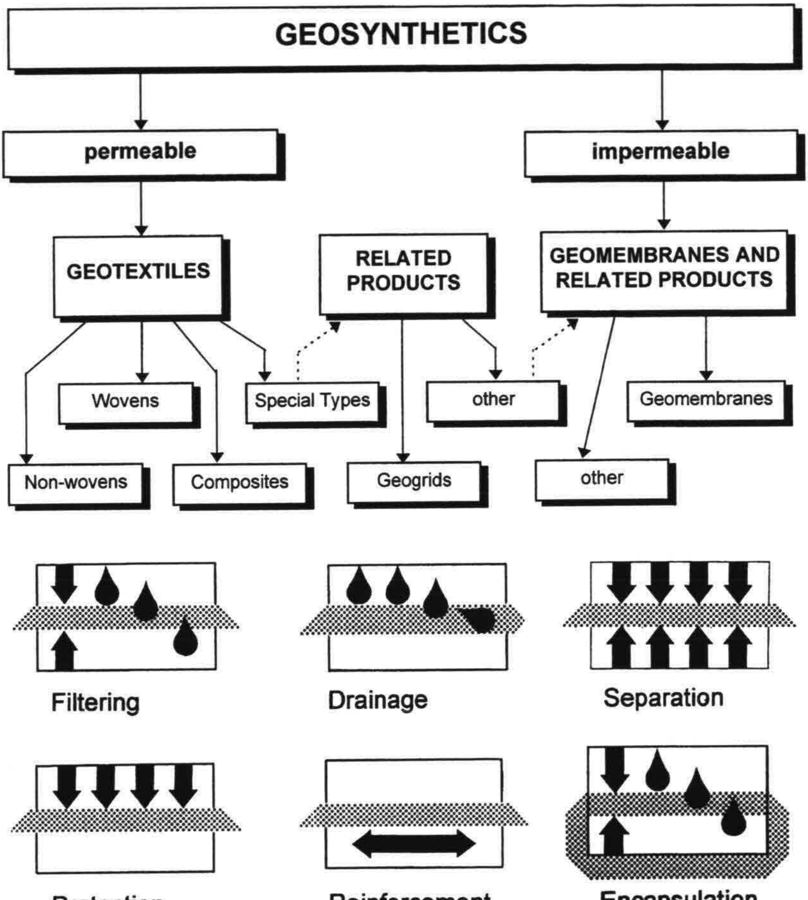


Fig. 3: Geotextiles in coastal engineering, classification and functions (from (4) and (24))

Areas of applications in the field of coastal engineering are for example (4, 7, 23):

- use as filter layers, e.g. in a revetment
- drainage layer in a dam
- separation layer, e.g. below a dam or a rubble mound structure
- protection layer, with e. g. a vulnerable geomembrane in the sealing of a structure
- woven fabric or geogrid as reinforcement of a filter mat or a steep slope
- encapsulation of sand and other materials

However, in a special application the functions of geosynthetic components (mainly geotextiles) can have different valences. A formal separation is difficult especially in such cases of application where different functions must be fulfilled by a geotextile element. Examples of this kind are:

- encapsulation of sand for coastal protection works where a geotextile must also fulfil the function of a filter
- reinforcement of a mattress of fascines, where separation of different layers and filtering of the existing subsoil respectively can also be considered as main function.

Constructions using geotextiles demand, therefore, detailed knowledge of the various functions which shall be attributed to the different geosynthetic components.

Both conditions during construction and application can be relevant for design. The conditions during construction including transportation, e.g. of a sinker mat to its final place, or dumping of stones on a filter layer, play an important role in the field of coastal engineering. According to these examples high requirements may result with respect to mechanical properties such as tensile strength and impact resistance.

During the lifetime of a structure a geotextile must often serve predominantly as a filter. Cyclic loads by waves and a possible migration of sediment particles in the horizontal plane of a geotextile must be considered in the choice of a suitable product.

Many of the recent applications of geosynthetics in the field of coastal engineering clearly demonstrate the advantage of an acceptance of even larger elongations of a geotextile. Since stresses are more or less important in the presence of flexible components and - on the other hand - stresses are often the cause of failure, for coastal engineering applications, as a governing principle, we should consider to think in terms of elongations rather than in terms of stresses, in my opinion.

In this paper some recent examples are presented.

Applications of geotextiles in Germany clearly show that besides conventional applications of geotextiles - e.g. geotextile filter layers are standard for the structural design of embankments - geotextiles fulfilling all the various requirements should be considered as a very important element in coastal engineering practice.

There are a lot of examples where the use of geotextiles leads to technical solutions which are better than conventional solutions also with regards to economy. Referring to long-term resistance of geotextiles investigations show a structural life time which certainly corresponds to the usually planned lifetime of a coastal work.

Standard solutions for the construction of a revetment as illustrated by Figs. 4 and 5 and to protect the transition between a hard structure and the erosive sea bottom may demonstrate some applications in hydraulic engineering in Germany.

3 General remarks on application of geotextiles

A variety of different applications can be found in literature.

The best review may be obtained looking at the Proceedings of International Geotextile Conferences. These conferences have a significance similar to the well-known International Coastal Engineering Conferences. The target group covers the entire field of civil engineering.

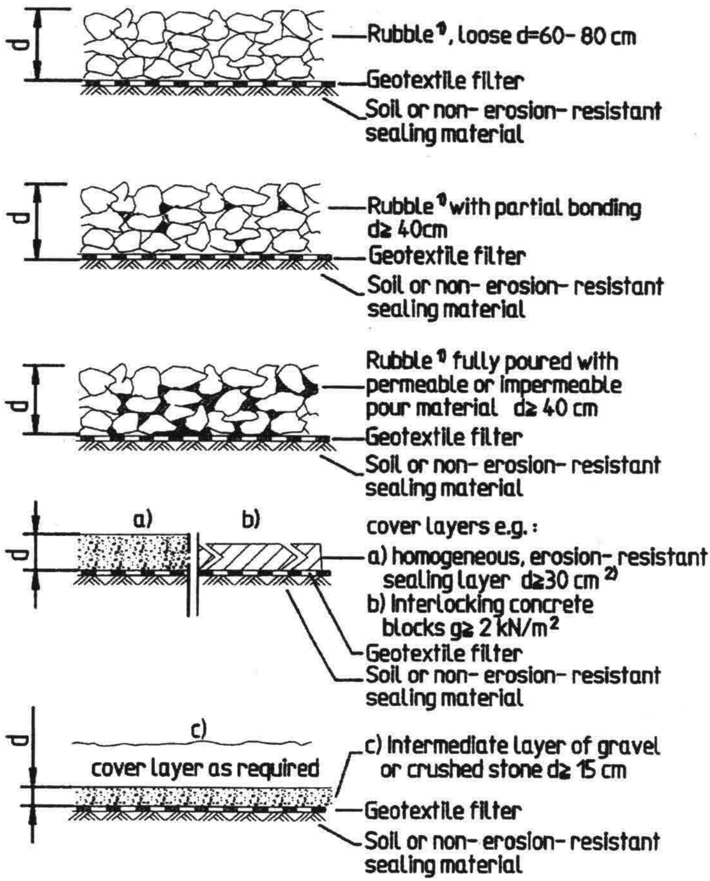
However, the contribution of coastal engineering is comparatively small.

Nevertheless, it should be noted that hydraulic engineering problems were the starting point of the technical development of geotextiles. Various other disciplines of civil engineering, e. g. the field of road construction or tunnel engineering were opened up later only.

During the extension of the Midland Channel revetments were constructed in Germany using coconut fibres for the construction of filter layers. Later on, geosynthetics were used to replace natural fibres. Woven fabrics, non-wovens and composites were used and further developed.

Considering the field of coastal engineering, it can be stated that filter layers in revetments using geosynthetics are very common today and, regarding the "Recommendations of the Technical Committee of Coastal Protection Works" (5) of the German Association for Earth Construction and Foundation

Engineering (EAK '93), most of the examples of dike and dune revetments were constructed using at least geotextile components.



1) Class II or III in accordance with the Technical Specifications for stone material used in hydraulic engineering
2) does not apply when asphalt construction methods are employed

Fig. 4: Standard solutions for revetment design using geotextile filters

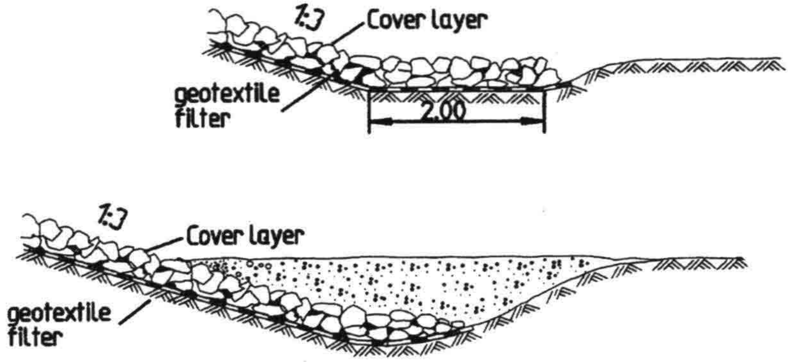


Fig. 5a: Standard solutions for revetment design (acc. to DVWK, 1993)

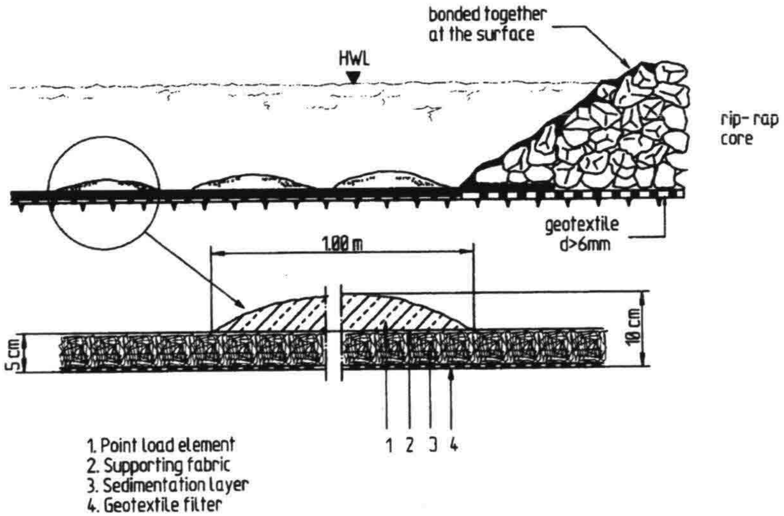


Fig. 5b: Stabilisation of a groyne at river Weser

(HEERTEN and ZITSCHER, 1984)

Advice and minimum requirements for the choice of a geotextile and the respective demands which must be fulfilled by a geotextile filter layer are included in EAK '93 (comp. also Fig. 4). Therefore, only some additional comments will be included here.

In the following, demonstrating the application of geotextiles, the use of bags and containers is treated as central issue. Solutions of this kind are almost neglected in the literature, in my opinion.

It seems to be difficult to succeed with technical changes in a discipline which is determined by tradition. In case of geotextile containers this might be due to the fact, that the solution is rather too simple to be acceptable as a technical concept. On the other hand side, these solutions offer really interesting possibilities for coastal engineering applications, if we consider the above mentioned aspects (2).

Besides some examples of "packed construction materials" such as bags, tubes and containers, the use of geotextile membranes will be illustrated. The application of membranes can be considered as a basis concept for coastal protection.

Also the so-called sand mat where sand is embedded between two layers of non-wovens and fixed by needle punching (in the same way of production as a geosynthetic clay liner) can be considered as an example of encapsulation (2). The sand is mainly used to weigh the geotextile. A sand mat of this kind can be excellently used as a foundation layer, e. g. for the construction of a breakwater. Besides this, the mat also serves as a filter.

4 Filter problems

In the broad field of coastal engineering applications a geotextile often has to fulfil the requirements of a filter.

Filtering means that the geotextile is able to retain soil particles moved by passing water in such a way that the resulting pressure head increase can be neglected.

These requirements are defined by the so-called characteristic (effective) opening size and the water permeability coefficient respectively (4).

It is often misunderstood that at least in a second or third priority a geotextile has to serve as a filter. Examples are the use of containers or even geotextile membranes to protect an eroding beach where, in the first instance, separation of different sand layers or the function of encapsulation seems to be important. Reference and advice of DVWK (Deutscher Verband für Wasserwirtschaft und Kulturbau e.V. - German Association for Water Resources and Land Improvement) and the filter rules as being recommended by the "German Technical Committee on the Use of Geotextiles" which are also schematically illustrated by Fig. 6 may demonstrate the interrelation of various functions.

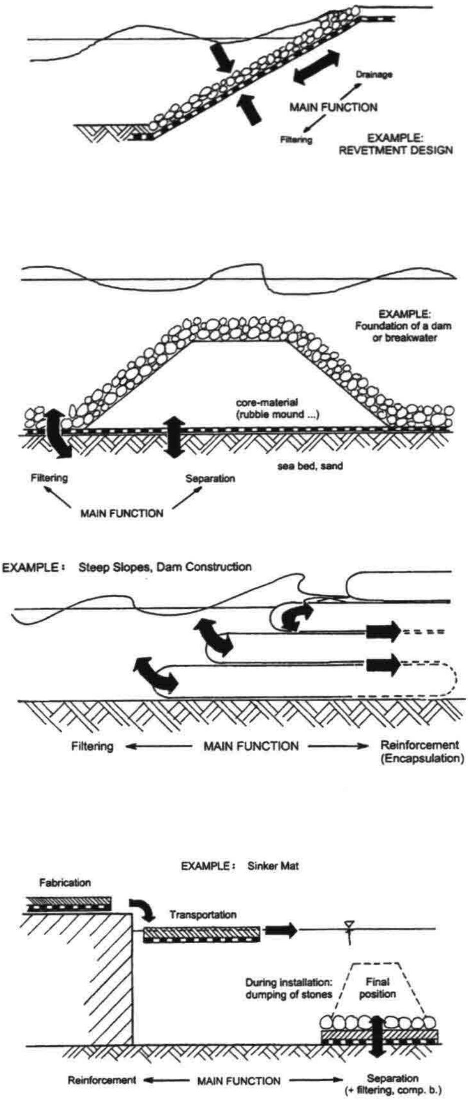


Fig. 6: Illustration of the interrelations of various functions attributed to geotextiles, schematic

During a special seminar on "Filters in Hydraulic Structures" organized by the Sri Lankan Coast Conservation Department (CCD) and the German Authority for Technical Assistance (GTZ) and held in Colombo (1989) the special requirements of mineral and geosynthetic filters were treated in detail. The importance of filter problems considering world-wide research is also clearly demonstrated in the Proceedings on the First Conference on Geo-Filters which was held in 1992 in Karlsruhe (3).

Despite the abundance of literature, the filter problem is controversially discussed even today. Surprisingly this particularly applies to the field of inland navigation. During the reconstruction of the Midland Channel in Germany, for example, geotextile filters were used in combination with special comparatively light concrete elements as a cover layer. Contrarily to other solutions and despite strong long-term loads caused by ship waves the revetments didn't fail during many years.

In spite of a lot of advantages in comparison with common solutions, considering both technical and economic aspects and specially regarding filter problems, the use of geotextiles is not always accepted.

Therefore, despite clear recommendations published by DVWK / DGGT (4), a subgroup of the Technical Committee of DGGT has started to reconsider the design rules but definitive results of this group are not available by now.

According to DVWK-standards it is common in the field of coastal engineering in Germany to prove the relation $O_{90,w} \approx d_{50}$. This equation is also used in the EAK '93 mentioned before (5). $O_{90,w}$ is the so-called characteristic (effective) opening size of the geotextile which must be determined experimentally by wet sieving under specially defined test conditions.

The value d_{50} corresponds to the 50% fraction of the grain size distribution curve of the soil to be filtered by the geotextile.

Practical applications show that thick, mechanically bonded (needle punched) non-wovens are particularly suited for the use under rough sea conditions.

Comparing the filter behaviour of mineral and geotextile filters, analogies were repeatedly formulated by HEERTEN (7, 8). These analogies show that the pores and the distribution of the pores have a dominant influence on the filter behaviour of geotextiles. Mechanically bonded non-wovens have a pore volume of approx. 90%. Even if a major part of the pore volume is filled by soil particles during the lifetime of a coastal structure, the permeability is not reduced to such an extent that pressure gradients within filter layer are remarkably increased (9).

Due to its three-dimensional effectiveness non-wovens are increasingly used in Germany compared with wovens, if we consider filter problems. Various applications of woven geotextiles can, however, be found in such cases when also requirements on the tensile strength must be attributed to the filter, according to the design of a structure.

It can be advantageous to combine wovens, non-wovens and geogrids when requirements of filter effectiveness have to be considered together with other functions of a geotextile in a special application.

Geo-composites are for example advantageous if we consider flow conditions within the plane of a filter, i. e. if we use a geotextile filter on a slope and soil migration has to be avoided in the contact area between filter and subsoil.

Pertinent problems were intensively investigated by MÜHRING (15) and the advantages of multi-layered geotextiles with a specially manufactured roughness layer were clearly elaborated based on a lot of tests.

Multi-layered geotextile filters can, after all, also be manufactured using different raw materials. Since the commonly used synthetic fibres have different properties (comp. (23)) we can systematically combine different raw materials according to the expected conditions during construction and lifetime, e. g. to combat the influence of radiation or abrasion by sand or ice, in order to achieve economically and technically optimum solutions.

5 Packed construction materials

5.1 Review and basic considerations

Different systems were developed following the principle of encapsulated materials (comp. Table 1) which can be used for various coastal engineering applications. For encapsulation of construction material ready made geotextiles and geogrids are used. As one of the major advantages these elements allow the use of local soil material. Also concrete or other materials can be used for special purposes.

Geotextile construction elements are exposed to various loads. Considering waves and breaking wave conditions they have to withstand wear and tear especially in such cases where the geotextile is not covered by stone material. With respect to abrasion by sand being moved with the water currents, geocomposites consisting of geotextile layers with different raw materials can be constructed to profit from the different properties and prizes.

- according to form and volume
  - hand bags (up to approx. 25 kg weight)
  - large bags (more than 25 kg, volume up to approx. $1m^3$)
  - geotextile containers (volume more than $1m^3$)
  - geotextile tubes (manufactured often endless)
  - gabions, baskets and mattresses
- according to material
  - wovens
  - nonwovens
  - composites
  - geogrids

  Remarks: combinations are possible to fulfil several functions simultaneously
- according to filling material
  - sand
  - stones
  - others

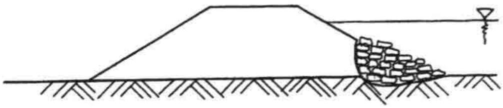
Table 1: Encapsulation of sand and other materials, specification of geotextile containers (from (20))

Most common in the field of coastal engineering are the hand bags used during disaster operations such as storm surges or high water events. More than 7 millions of bags were used during the high water disaster at River Oder this year to give a recent example of application.

In the following, as a generic term for all the different solutions, we use the term container. The most important applications are illustrated in Figs. 7a and 7b. General aspects for assessment and dimensioning are listed in Table 2.

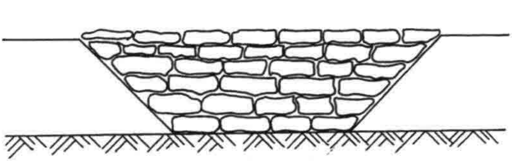
SAND BAGS AND CONTAINERS | EXAMPLES

- Locking of gaps and damages in embankments during high water attack and storm surges



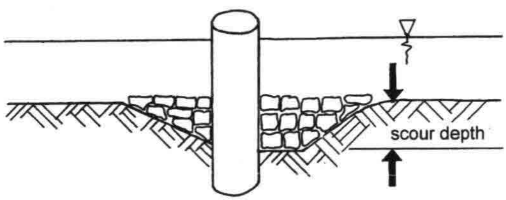
temporary protection using handbags

- Closing of dike or dam during construction



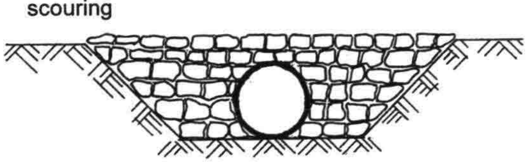
placing of containers, hydraulic filling of tubes

- Protection and securing of structures exposed to wave and current action



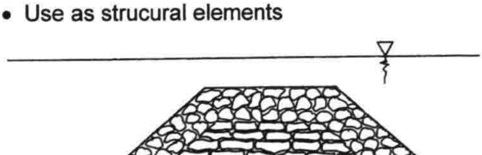
protection of a pile construction

- Protection of pipelines and traffic tunnels against scouring



protection of a culvert installed in a trench

- Use as strucural elements



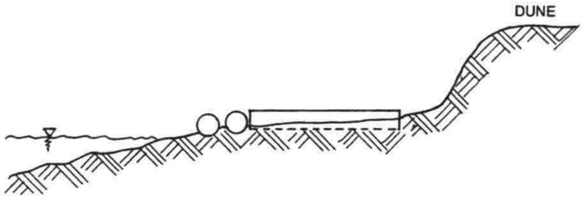
packing of core material in dikes and dams construction elements of groynes and polder fences

Fig. 7a: Illustration of possible applications of sand bags and containers

TUBES AND MATRASSES

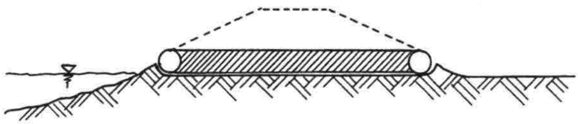
EXAMPLES

- Stabilisation of sandy beaches, coastal protection



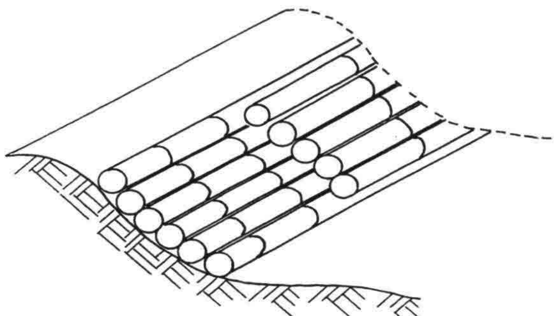
geotextile tubes positioned (alongshore and perpenicular) on the beach

- Fixing of the limits of a sand fill-up



construction of a sea dike using hydraulic transport of sand in pipes

- Construction of coastal structures, such as groynes and revetments



protection of an eroding beach by tube or matress

Fig. 7b: Illustration of possible applications of tubes and mattresses

• GENERAL REQUIREMENTS ON GEOTEXTILE AND FILLING MATERIAL: – essential function: Encapsulation of sand temporalily (e.g. during transportation) or permanently (e.g. inside a dike) – the container (and the filling material) are a structural element of a coastal structure. Additional functions, such as filtering and / or protection, reinforcement must be considered – the container must have a certain weight to withstand waves and current action – additional requirements on the geotextile must be considered to guarantee the filtering function, abrasion resistance and others • MANUFACTURING AND FILLING REQUIREMENTS ARE STRONGLY INFLUENCED BY FILLING PROCESS AND MAY BE DESCRIBED BY – mechanical behaviour such as σ - ε characteristics, maximum elongation – tensile strength, bursting resistance, including seams and joints – resistance against high temperatures (occasionally, e.g. when filling with asphalt) – filtering characteristics, permeability and soil retention capability, specially in case of hydraulic filling of a container • DEMANDS DURING TRANSPORTATION AND INSTALLATION – depending on location and kind of the used containers – essential points of interest are for example lifting of the container by (specially designed) equipment (protected edges of gripping device) – crashes of falling containers and stone dumping

Table 2: Encapsulation of sand, general aspects for assessment and dimensioning (from (20))

5.2 Sand bags

Well-known are sand bags which are used for dike construction and repair. In addition to natural fibres synthetic material is increasingly used for manufacturing of the bags. Two kinds are in use considering practice. Firstly the "handy" bag with a filling weight of approx. 25 kg and secondly the "large" bag with a filling volume of approx. 1 m^3. In the first instance, the handy bag is in use for the repair of dikes after high water and wave attack whereas large bags are also used as construction elements (20).

To close the bag, two bands are sewn to the geotextile. They must have the same strength and durability as the material of the bag.

In case of a disaster the bags are ready at hand. They can also be stored in filled condition.

Additional requirements must be fulfilled by larger bags. Especially, we have to prove the material's durability against loads resulting from transport and installation (comp. Table 2).

The geotextile and the filling material must be brought into line in such a way that the filling material doesn't pass the pores of the geotextile due to currents. The bag must be handy and must have proper friction behaviour (also during stockpiling). These requirements can be fulfilled by both woven and non-woven materials where the latter show advantages considering flexibility and friction behaviour.

5.3 Sand-filled tubes

Using tubes as a geotextile container the filling material is completely encapsulated and the almost circular sand-filled tube may be considered as an independent construction.

According to EAK '93 (5) we distinguish between the Danish and the East Frisian system.

The Danish methods includes a water tight (PE)-film which is manufactured together with the geotextile (woven PE- or PP-fabric). The tube is filled in its final position. The water is being pumped into the tube where sand is added by means of a funnel. Whilst the sand is retained inside the tube the water is drained off through a pipe at the end of the tube.

In contrast to the Danish method the East-Frisian method disclaims a synthetic film using a fabric only. Up to a diameter of approx. 2.0 m the tubes are manufactured both without seams and webs.

A suction dredge is normally used to fill the (permeable) tube, which can consist of a woven or a non-woven geotextile. After deposition of the sand the water is drained through a pipe at the end of the tube. According to the actual demands the geotextile is dimensioned in such a way that the sand is retained by the geotextile and only water which is pumped into tube (together with the sand) can pass.

The height of the filled tube is approx. 85% of the theoretical (circular) diameter of the tube.

After execution of the filling process, e. g. in connection with the replenishment of a beach, the tube can be covered by stones and may serve as a part of a revetment. Such a sand-filled tube may also be used as a core of a groyne or part of a dam.

Both methods, the Danish and the East-Frisian, allow an immediate economic and quick filling where we can use local and often even silty soil (comp. (14)).

5.3.1 Example no. 1, Use of sand-filled tubes during dike construction

Since 1967 sand-filled geotextile tubes are used at the German North Sea coast (6) especially for shore protection purposes on the East-Frisian islands. A more recent application is the use of geotextile tubes during the construction of dikes in the "Leybucht", State of Lower Saxony (11). According to the technical developments geotextile tubes can be easily filled having single lengths of 400 m. By means of sewing on additional geotextile sections, jointless constructions of any length are possible today.

From the experience of the construction site at "Leybucht" it should be noted that not only woven fabrics (high tensile strength) but also non-wovens (large elongation) can be filled without major difficulties (11).

Considering non-wovens, it might be seen as a disadvantage compared with wovens, that non-wovens can have a much higher weight (due to soil migration into the pores of the geotextile and content of water) which makes it sometimes difficult to change the position of the tube prior to filling (specially during rainy season and softening of the subsoil). Advantages of a non-woven, needle-punched geotextile are the prize and the robustness against mechanical loads during both construction and lifetime of the structure.

5.3.2 Example no. 2, Kirra groyne, Australia

Another example of the use of packed sand is the protection scheme of the Gold Coast at Kirra, Australia. Gold Coast is an important recreation area with approx. 30 km of sandy beaches. Since tourism is the living basis for many people in this area, a comprehensive programme was elaborated to protect the sandy beaches. Three groynes were constructed using sand-filled geotextiles.

The largest groyne was constructed in North Kirra (Fig. 8). The groyne has a length of 110 m, the height is 5 m. At the base the width is 12 m.

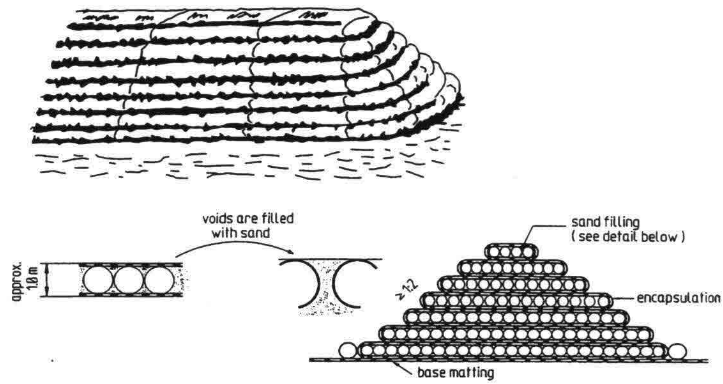


Fig. 8: Construction of a geotextile groyne at Kirra, Australia (from (4))

The groyne was constructed using geocomposits. One layer serves as an impermeable sealing. The second layer is a robust geotextile. A non-woven (needle-punched) was used, which covers the inner tubes of the groyne.

According to up to now experiences, geotextile groynes can be considered as a realistic alternative to conventional groynes, which are mostly constructed using stones. Advantages are becoming evident specially in areas where large stone material is hardly available and stones must be transported over long distances and at high costs.

5.4 Containers filled with sand

Geo-containers can be manufactured in any form and of any volume, in principle (comp. Table 1). They can be filled directly at the construction site. Coarse stone materials as well as sand can be used to fill the containers. The material is mostly available at low cost.

Besides woven materials with high tensile strengths, non-woven materials also can be used. These materials are able to follow all the irregularities of the bottom due to their flexibility where the activated forces within the geotextile can be more or less neglected. There is also the possibility to combine both the flexibility and the strength within a two- or multilayer-product. A non-woven product and a geogrid or a woven and a non-woven layer can be, for example, bonded together in the factory.

In 1986, huge sand containers consisting of mechanically bonded (needle punched) geotextiles were tested under severe conditions (North Sea, Island of Nordstrand). These containers had a filling volume of approx. 20 m^3. The weight of the filling material was approx. 30 t (4). The only weak point of these geo-containers consist of the seams, a problem which was, however, solved after some testing and using high strength yarn.

The use of containers filled with sand has a lot of advantages compared to other technical solutions and presents itself for example for construction of dams or dikes (especially for closing a gap) or the filling of a deep scour.

Advantages can be as follows:

- Controlled placement is possible even during comparatively strong currents and in large water depths
- During placement of a filled container there are almost no losses of material (compared with free-fall dumping of material)
- Since the filling material sand is locally available, costs for material and transportation are reduced.

If we look at the literature there are many possibilities of application (comp. Fig. 5), e. g. as a protection measure for earth structures, for culverts or foundation piers.

If we consider the area of coastal engineering, the use of containers should be considered for the core construction of dikes and dams as mentioned before.

Recent examples of coast protection applications are illustrated in chapter 6.

6 Recent developments for the use of geotextiles for coastal protection

6.1 Restoration of a sandy bar using geotextile containers, Island of Sylt/Germany

Various applications for the use of geotextile containers were recommended to encounter coastal erosion problems on the Island of Sylt but, up to now, only the restoration of a sandy bar was executed. This was done at the village of Kampen, where the natural sandy bar doesn't exist due to various reasons (see site plan, Fig. 9).

The area of Kampen is characterized by the fact that due to the absence of the natural bar the beach is strongly attacked by waves passing the existing gap more or less unhindered. One must expect that a restoration of the bar, especially by increase of the crest height, will lead to a decrease of the inshore wave energy. According to theoretical and experimental investigations (17, 21) even a slight increase will lead to a remarkable reduction of the inshore energy. A reduction of sediment transport can be correspondingly expected (Fig. 10).

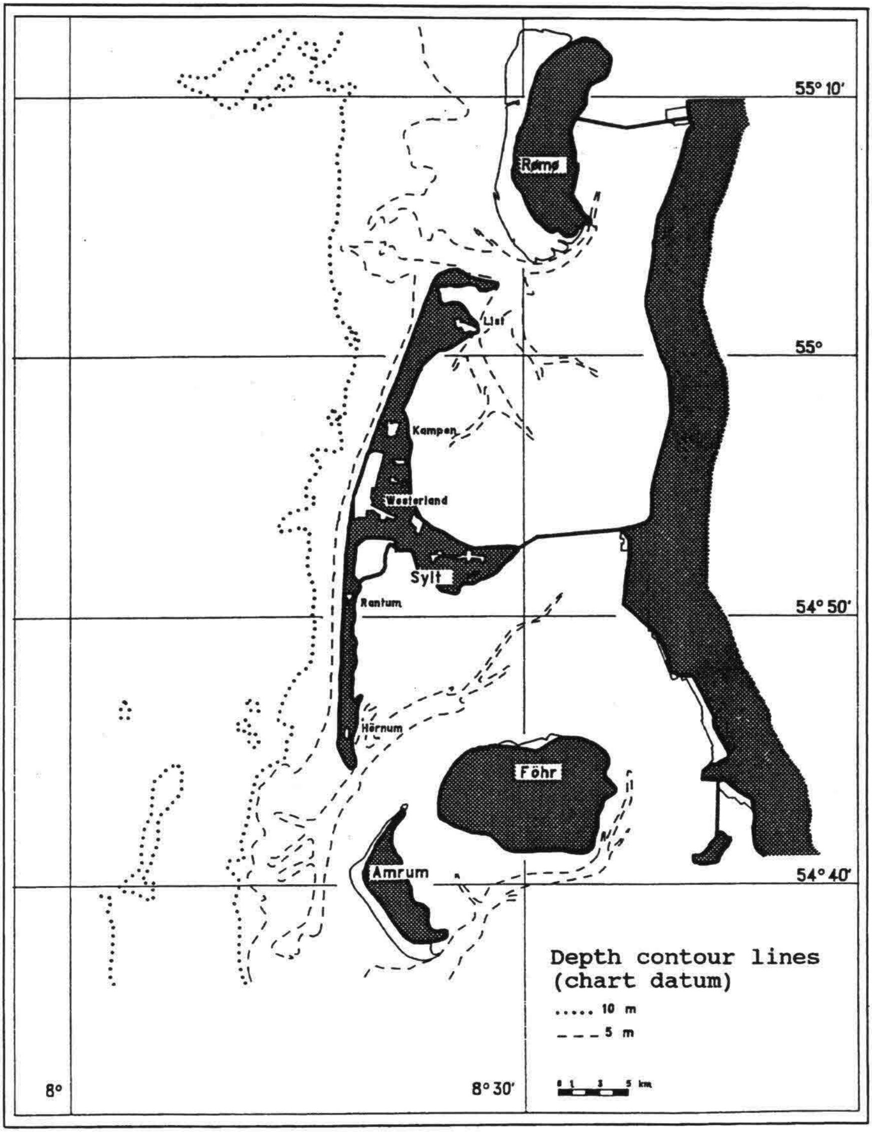
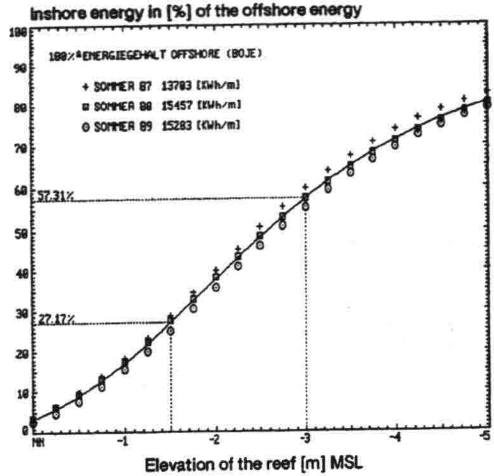


Fig. 9: Island of Sylt/Germany, site plan



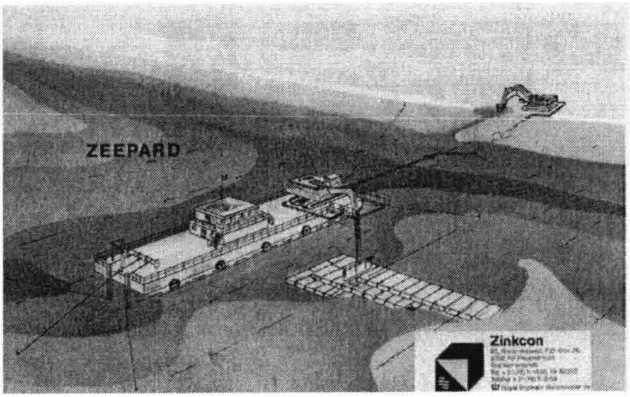


Fig. 10: Restauration of a sandy bar at Kampen, Island of Sylt, using geotextile containers (from (13))

Restoration of the bar at Kampen was executed in 1990. Different materials were used to gain experiences for the use of large geotextile containers exposed to waves and sand without further protection and including also the limits of application.

Unfortunately, the effects of the geotextile bar cannot be finally analyzed up to now. Since beach nourishment measures were executed at Kampen at the same time as the restoration of the bar was performed, the artificial bar system was immediately covered with sand. Influences of the bar and beach replenishment are, therefore, superimposed and the effect of the containers on expected wave attenuation and sand transport respectively cannot be assessed without doubt.

Other fields of application were assessed during a comprehensive research programme "Optimization of coastal protection works, Island of Sylt". The research group representing various German universities as well as governmental institutions specially considered the use of containers for construction of an underwater terminal groyne at the southern tip of the island and the use of containers and geotextile membranes to protect the lower parts of the dunes. There are many possibilities also regarding the protection of the tidal channels and eroding embankments in the north and the south of the island. For example, geotextile containers were used for the reconstruction of the harbour of List, a village in the north of the Island of Sylt.

6.2 Use of geotextile membranes, example Island of Sylt/Germany

The term optimization of coastal protection measures may be interpreted in such a way that the costs for construction maintenance and repair are minimized where negative influences on the adjacent stretches of a beach must be included. Optimization which includes the reduction of negative consequences of a technical measure needs every efforts as the example of the Island of Sylt clearly demonstrates. As mentioned before, one of the concepts was to construct a geotextile barrier at the southern tip near the village of Hörnum (situation comp. Fig. 9).

Call for tenders for this coast protection measure was in 1990. Geotextile containers with a volume of 5 m^3 of sand were scheduled by the governmental authority where different geotextiles should be used for comparison.

It was planned to use a geotextile membrane as a foundation of the containers to prevent scouring.

As a result of the tendering and due to unexpected high costs this coast protection measure was not executed up to now (considering the technical concept it should also be mentioned that the project, especially the foundation, was controversially discussed within the research group despite a lot of advantages compared with other measures). Anyhow, the use of membranes offers itself, e. g. to close a gap during dike or dam construction. Flexible non-woven geotextiles were successfully used, for example, to separate coarse stone material and fine sand during construction of a stone dam (24) and for the construction of a light revetment in Germany (13).

According to the various disadvantages of hard structures, which were briefly outlined before, the Master Plan for the Island of Sylt (1) doesn't allow any hard structure today if any possible. Beach nourishment is executed since many years on the island and hard structures are only used to support the stability of sandy depots and to prolong the life time of an artificial beach.

Both the technical measures of the bar restoration mentioned before and the planned construction of a geotextile dam in the south of the island can be assessed in this sense, i. e. to minimize the losses of sand and thus to minimize the overall costs of a coastal measure.

Fig. 11 shows the concept of light revetment using a geotextile membrane placed within an artificial sandy depot. In the lower part of Fig. 11 we can see the membrane after a severe storm.

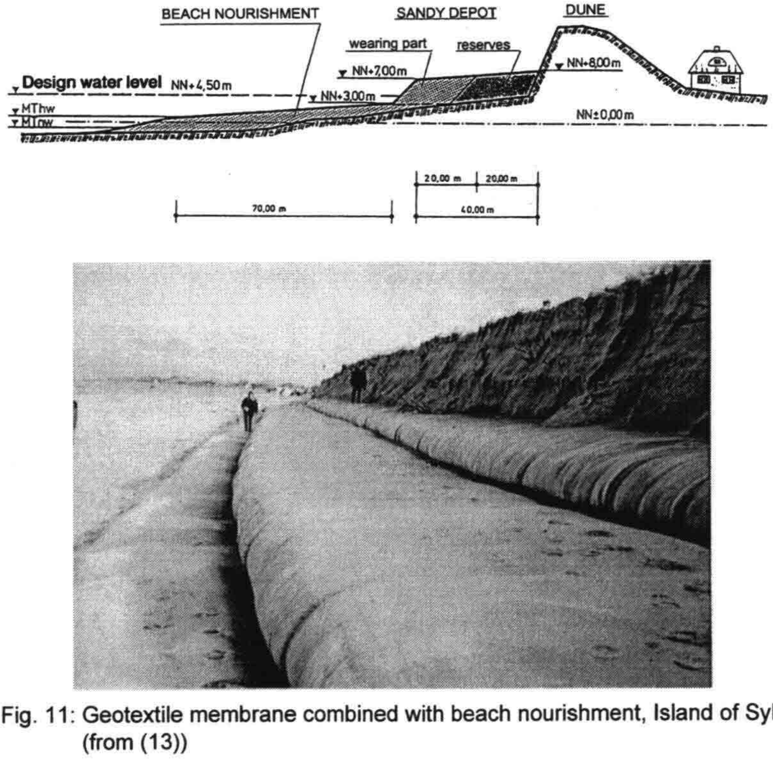


Fig. 11: Geotextile membrane combined with beach nourishment, Island of Sylt (from (13))

This measure was planned and executed to protect a historical building at Kampen which was strongly endangered by erosion. One can, of course, consider such a structure also as a special example of encapsulated material, e. g. packing of sand.

Since the dividing plane between the protection part and the reserve part of the sandy depot where the geotextile membrane is placed cannot be exactly calculated and, hence, the plane is a fictitions border only, the geotextile can be considered as a safety belt within the depot. In case of a storm surge condition being combined with strong waves leading to unexpected losses of sand the geotextile has to serve as a revetment for a limited time of severe hydrodynamic loads.

After a storm event the membrane must be, however, covered again with sand.

Long-term experiences are not available up to now but experience from a severe storm event in 1994 (comp. Fig. 11) confirms the basic concept.

During this storm event the light revetment was discovered without any visible damages of the geotextile. It may be noted that, even during large-scale stability investigations performed in the Large Wave Channel in Hannover (GWK) it was impossible to destroy the geotextile membrane and its general ability to serve as an stability element within a sandy depot was clearly demonstrated.

Nevertheless, investigations on the use of geotextile containers under natural conditions are planned by IWR to gain more experience specially with respect to the influences of raw material and different kinds of manufacturing on the long-term stability under rough conditions.

7 Summary and concluding remarks

Despite the progress which can be noted regarding the development of theoretical approaches in coastal engineering and despite increasing knowledge of the physical interrelations between hydrodynamic loads and the reaction of coastal structures, the field of coastal engineering is still an empirical discipline in many respects.

It is almost impossible to forecast the interrelations between the hydrodynamic input (mainly waves) and the reactions of a coastal structure and/or the erosive sea bottom. Especially, the influences of coastal structures on the morphology of adjacent stretches of a beach can be estimated only insufficiently today. This often leads to conceptual mistakes.

Corresponding to the sand budget of an eroding beach, "passive" coastal structures are useful for local protection only. The influences of a hard structure fixing the beach and limiting its natural changes (dynamic equilibrium of the beach profile) are often ignored. Generally, the weakening of longshore sediment transport often caused by a structure leads to the phenomenon of downdrift erosion, with the area of erosion depending on the directions of waves and currents and their seasonal variation.

Structural failures can often be related to an insufficient design of filter layers. If we consider a sandy beach characterized by a negative sand budget, erosion cannot be prevented by any structure alone. Beach nourishment and beach replenishment measures are therefore increasingly executed to balance the natural losses of a beach. This is often done in combination with a conventional coastal structure, such as groynes and breakwaters. Considering both, conventional (hard) structures and beach nourishment measures, geotextiles play an important role according to many year's experience.

Geotextiles are used since a long time regarding the functions filtering and reinforcement. They may be considered as a standard solution for the design of a revetment today. The same applies for geotextile bed protection mattresses in the field of coastal engineering.

Looking at recent applications, geotextile constructions are more and more planned and executed using containers where the function of encapsulation of sand and other materials is the dominant function.

Examples showing this tendency in coastal engineering are the restoration of a sandy bar-system offshore the village of Kampen, Island of Sylt using large geotextile containers and - at the some location - a geotextile revetment to protect a historically valuable single house.

The idea to pack construction materials into a geotextile, i. e. to use the geotextile mainly as a packing material, often shows essential advantages compared to conventional solutions in the field of coastal engineering. Geotextile constructions or structural elements can be, for example, easily removed in case the structure doesn't lead to the planned function or unexpected farfield effects on neighbouring stretches of a sandy beach cannot be tolerated. On the other hand, in case of positive effects according to functional planning, a geotextile structure can be integrated into a conventional coastal structure. This can, for example, be done by covering the geotextile bags, tubes and containers by stone material, to prevent any danger of vandalism or to protect the geotextile against scrubbing by moving sand. Covering of the geotextile structure will not generally reduce the necessary flexibility of a coastal structure as a whole.

Especially the transition areas between stiff parts of a hard structure and the erosive sea bed must be carefully designed. Effects of scouring due to sea waves and currents can be minimized using flexible geotextile constructions. In contrast to a conventional hard structure a geotextile structure can easily follow the morphological changes of the bed without further damage.

From the above shown examples illustrating various possibilities of application in the field of coastal engineering it becomes clear that the functions which can be attributed to a geotextile have different valences for any special case of engineering interest. It is, however, often difficult to separate single functions, and usually more than one function must be assessed.

The examples shown in this paper are applications where the function encapsulation dominates but other functions, e. g. reinforcement and filtering cannot be neglected.

Therefore, any construction using geotextiles requires detailed consideration of functions which shall be attributed to the single components. The conditions during construction as well as the conditions during the structural lifetime can be decisive in this respect.

Conditions during execution of coastal works often play an important role in this context. Stresses may be caused by placing or dumping of stone material on an unprotected geotextile layer. This requires certificates on mechanical properties, such as tensile strength as a function of elongation. Considering the filtering effectiveness the rough conditions at the sea side causing cyclic loads and flow conditions must be taken into consideration as well as corresponding migration of soil particles in the boundary layers between geotextile and soil.

The variety of possible applications (also together with conventional technical solutions) clearly demonstrates, that geotextiles can be seen as an important structural element in coastal engineering practice. There are many examples showing that the use of geotextiles can lead to solutions which are even better than a conventional solution with respect to technical design and economy.

Geotextile structures and geotextile components within a coastal structure should be considered as an alternative also because of their flexibility. However, maximum care must be ensured during installation. Considering the durability of geotextiles we know from scientific investigations that geotextiles can have a lifetime which definitely corresponds to the planned lifetime of conventional coastal structures.

The filling of geotextiles, i.e. encapsulation with sand which is available on site often leads to low cost variants of technical design. Another positive aspect can be the comparatively short time of installation. Without major problems we can further install additional layers of containers, if necessary, in the same way as it is possible to remove a geotextile structure.

It should, however, be mentioned that compared to conventional methods geotextile structures are prone to mechanical stresses, especially vandalism. Special care must be guaranteed in areas with public access, e. g. by covering an unprotected geotextile structure by sand.

8 Acknowledgements

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Sinkholes Problem Happened in Access Road Behind Sea Wall

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Abstract

After several typhoons attcked in summer 1994, severe damages appeared as sinkholes beside the access road or sinkholes in the middle of access road was observed. The sinkholes reappeared some time after the holes were filled. Through the field investigation especially focused on the sinkholes appeared in the middle of access road, it is believed that piping phenomena is the major reason for the damage. The reason for the sinkholes beside the access road is supposed to be the excessive pressure resulted from the so called venting effect which was first mentioned in similar case happened in Tripoli Harbour NW Breakwater before.

1 Introduction

On the west side of the reclamation area of the planned Chang-Hwa Industrial park which is located at middle west coast of Taiwan, several kilometers of seawall were constructed up to a preliminary elevation in order to protect the reclamation area, and an access road just behind the seawall was built for transporting the construction material used for the subsequent seawall construction in the south side. After the attack of several typhoons during summer of 1994, severe damage apeeared as sinkholes on the access road. Besides, it was observed that the size of sinkholes gradually increased. Although the sinkholes was then filled with some gravel, the sinkholes still reappeared after some time. Therefore, the planned pavement of the access road was not able to be completed. In order to understand the actual reason for the appearance of sinkholes and to prevent this incidence in the future, Tainan Hydraulics Laboratory was asked to investigate the formation of this problem and to suggest a feasible countermeasure in the last stage of work.

2 Problem Description

2.1 Site Visit

A site visit was arranged in November 1994. It was found out during the site visit that there were about 20 sinkholes in 2 kilometer of access road. Two types of sinkholes could be characterized. The first type of sinkholes appeared as remarkable holes underneath and beside the drain ditch between the seawall and the access road that made the drain therefore useless. It was observed apparently that the plastic filter laid between core and reclamation fill were torn off, the reclamation fill under the drain and access road were therefore piped out from the torn holes. The second type of the damage appeared as sinkholes in the middle of the access road, and there were no apparent outer appearance showing the cause of these holes. Photo 1 and 2 are the pictures about the two types of sinkholes taken during the site visit. On the other hand, it was also found out during the site visit that the two types of damage appeared exclusively in different section of the access road. It is afterward found out that the border is just about the border of two different types of sea wall cross section. The core of the first type of sea wall structure is filled with sand and the second type of sea wall is the rubble mound type sea wall. Typical section of the two types of cross section are schematically sketched as shown in Fig. 1.

2.2 Preliminary Investigation of the Reason behind

The formation of the first type of sinkholes is with no doubt that the reclamation fill underneath the access road was piped out through the torn plastic filter. There was however no obvious explanation for it during the site visit. On the other hand, it can be imagined that the formation of the second type of sinkholes should also base on similar reason, however, it still stays a guess without seeing torn plastic filter under the access road, and not to mention the actual reason for it. Through some literature survey, it is found that similar incident also happened before on the Tripoli Harbour NE Breakwater of Libya, and it is mentioned by Barony, et. al. (1983) that the so called venting effect plays an important role in the formation of sinkholes . It is thought that the two types of sinkholes might also due to the venting effect after the cross section of the sea wall was compared with the Tripoli case.

3 Field Investigation

Learned from the experience of Tripoli Harbour NE breakwater, the aforementioned venting effect was suspected to be the reason for the sinkholes investigated. However, except for the first kind of sink holes that torn plastic filter was exposed on the surface, it is still purely a guess, although seems quite reasonable, to say that the second type of sinkholes was also formed with similar reason without observing some torn plastic filter. Therefore, it is necessary to excavate the second type of sinkholes on one section of access road in order to find out if the plastic filter laid between the rubble mound core and the reclamation fill was really torn off by some external force. On the other hand, in order to further investigate the origin of the external force exerted on the plastic filter of the second type of sinkholes, field measurement was therefore planned. However, according to the design sketch of the first type of sea wall, the suitable location for instrument installation is under the crown wall and will not be realistic to dismantle the crown wall for that purpose, besides, it was quite possible that excessive pressure caused by venting effect or other relevant phenomena is the major reason for the first type of damage. Therefore, the field investigation including the field measurement concentrated only on the second type of damage.

3.1 On Site Excavation

Sinkholes A, B, C and D of the second type are chosen for the excavation since

7 sinkholes appeared in about 50 meter of length which indicated a critical section. In order not to ruin the plastic filter by mistake, detailed excavation near the location of the plastic filter was done by worker with shovel, and excavator with 320 HP capacity was used only for surface excavation. It was not surprisingly found out that the plastic filter under the access road were actually torn off and the reclamation fill were then piped out through the torn holes. Several common features were observed and described as follows:

(a) According to the original design, the plastic filter should be placed between the rubble core and the reclamation fill, it was found out after excavation that there did exist much fine material in the core which supposed to be consisted of cobbles with diameter larger than 10 cm.

(b) The slope of the plastic filter was not 1:1.33 any more, the slope was steeper in the lower part of the plastic filter. Besides, the plastic filter was torn around the level with water mark.

3.2 Field Measurement

Even the torn holes were found on the plastic filter, the origin of the external force which cause the damage was still uncertain. Field measurement was therefore planned, and instrument was installed directly after excavation. The object of the field measurement was to investigate the influence of wave pressure arisen in front of the sea wall and to know whether it is possible or not that excessive pressure resulted from possible venting effect can penetrate the core material and damage the plastic filter.

3.2.1 Instrument Installation

Four pressure transmitters were mounted just behind the core and in front of the replaced new plastic filter layer, the elevation of the four pressure transmitters are about 1.8, 2.4 , 3 and 4 meter under the suface of access road. One additional pressure transmitter is installed above the plastic filter layer and about at the same elevation as the deepest pressure transmitter in front of the plastic filter in order to have some idea about the pressure head difference between the two sides of plastic filter layer and see whether the pressure head difference also plays some roles in tearing the plastic filter layer. The whole arrangement came from the idea that venting effect might also be responsible for the torn plastic filter since it was thought during the planning stage that the sea wall structure is constructed just the same as the original design. Although it was then thought during excavation that the venting effect might not happen for the rubble mound sea wall since there existed much fine material in the core material which will most probably damp the pressure even if some venting effect appeared at the front side of the sea wall. However, to see is to believe, it is still worthwhile to do so. The pressure transmitters are fastened on the stainless rod which was mounted under the bottom of the drain ditch.

The pressure transmitters used are E713-023-B18R(2.5 bar) and E713-023-B19R(4.0bar) manufactured by french BOURDON SEDEME company. Certain process was done to improve the waterproof of the pressure transmitters. Cable length is 35 m and the output signal is 4-20 mA. R-C circuit is installed in order to transfer the current signal into voltage signal which can then be recorded by computer through A/D converter.

3.2.2 Arrangement of the Data Acquisition System

The object of the field measurement is to investigate whether venting effect appear or not. If such effect do exist, it is imagined to appear with the form of high pressure and short duration and this excessive pressure should be much higher than the static water pressure and the frequency should also higher than the wave frequency. Therefore, the data aquisition program was so designed that pressure data will be taken in 100 Hz frequency for 10 second once such excessive pressure appears, and the threshold of pressure is taken as 0.5 bar. In normal situation, the pressure record will be taken every 10 second. The data acquisition system and the personal computer is put in a 20 ft container about 20 meter away from the D point. However, due to the fact that it is full of risk of keeping all these equipment in the container since many stealth happened in this area, it is finally decided to keep only the cable outlets of pressure transmitter in the container during normal days. Therefore, the PC-based data acquisition system was transported to site before severe sea state was expected, like for example, typhoon period or during winter monsoon seaon, and data were then collected.

3.2.3 Field Data

The installation of pressure transmitter was settled down on May 23rd, 1996, several measurement was done during 1st and fifteen day every month of moon calendar which are the period with higher tidal level than normal. No special phenomena was arisen. Fig. 2 is one record between July 18th and 19th, 1996. It shows that the pressure variation was caused by tide, and the pressure head difference caused by the plastic filter is not big. The highest pressure head counted at the lowest pressure transmitter is about 1250 mm. The most important record was obtained between July 31st and August 2nd, 1996 during the attack of Typhoon Herb. This typhoon caused severe damage for the whole island. Fig. 3 is the measuremnet during that period. It seems that the venting effect did not appear in this type of structure even during the attack of typhoon, although the pressure did vary significantly during the highest tidal level which is believed to be the period with storm surge. However, no extraordinary excessive pressure was observed. Therefore, it can be preliminary concluded from this result that the damage of the plastic filter was not caused by some penetrated high pressure due to severe sea state in front of the sea wall.

4 Inference of the Cause

Since no extraordinary high pressure was observed during the typhoon period, it should be some other reason responsible for the damage of the plastic filter. There was really some hint during the excavation of the second type of sinkholes. According to the original design of the sea wall structure, the slope prepared to put the plastic filter should be 1: 1.33. However, the plastic filter did have a steeper slope especially under the water mark which can be clearly seen during excavation, and the torn place also appeared around the water mark. It is also observed that the core material contained too much fine material instead of the designed cobble stone with diameter bigger than 10 cm. Therefore, the reason of the damage could possibly be drawn according to the present investigation as follows:

(a) Since it was found that there is a layer with fine material mixed with some gravel in front of the plastic filter, the fine material could be piped out through core due to piping phenomena during ebb tide period. Therefore, when this layer sank due to lost of fine material, the plastic filter sank also due to the loading above and the lack of enough support underneath, and the profile of the plastic filter was then getting steeper.

(b) This kind of deformation continued until the critical point is reached that the plastic filter could not resist any further deformation and the plastic filter was then torn off. The reclamation fill will then piped out through the torn holes of plastic filter. The sinkholes was therefore formulated in the middle of access road.

5 Concluding Remark and Discussion

It was after excavation found out that the second type of sinkholes appeared in the middle of access road was due to piping phenomena through the torn holes on the plastic filter. Based on the result of the field measurement through typhoon, it is pointed out that the venting effect might not be the cause of the torn holes. On the other hand, it was observed during the excavation that the fine material mixed with some gravels was within the core which supposed to be consisted of only cobble larger than 10 cm. This kind of arrangement is actually not based on the design and might be some expediancy during construction stage without careful evaluation. This might result piping especially during ebb tide and fine material could be piped through the core material. According to the observation during excavation work, when the fine material was piped out gradually, the profile of the plastic filter laid on this layer was also becoming steeper and steeper, and the plastic filter suffered tension from deformation. Once the deformation is over the limit of the material, the plastic filter was then torn off, and reclamation fill behind it could be piped out and then resulted in the sinkholes in the middle of the access road.

Although the origin for the present case is most probably from the unappropriate arrangement of the layer contained fine material mixed with some gravel, it is still questionable if the original sea wall design could be better since the original design is similar to the breakwater design of the Tripoli Harbour except that the latter used larger material as core. If such designed cross section are still to be considered, it is necessary to check through hydraulic model test whether venting effect will appear or not.

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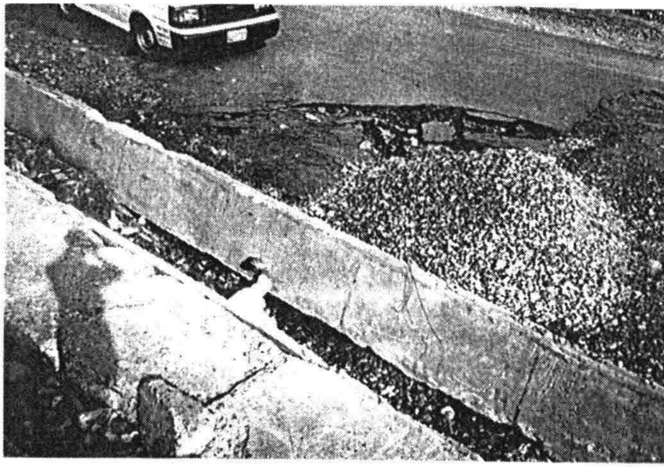


Photo 1 The first type of sinkholes

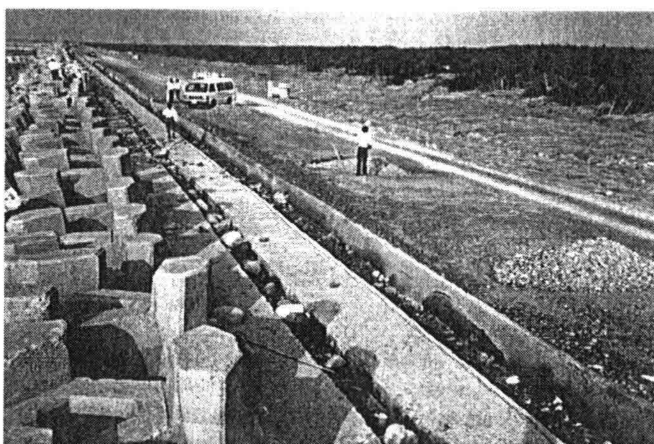
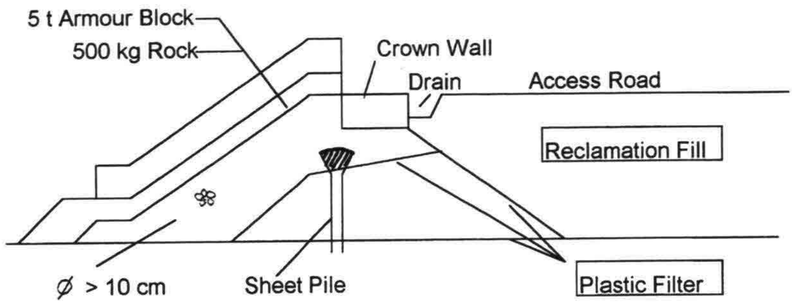
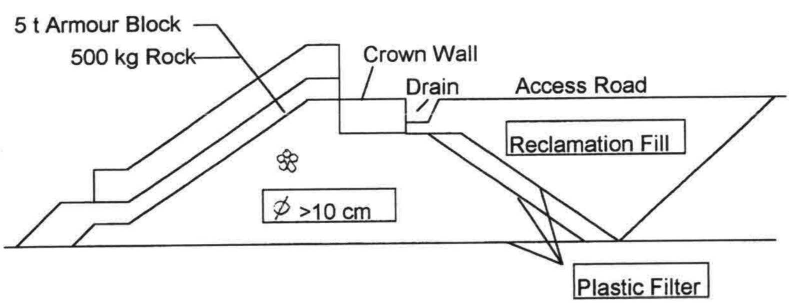


Photo 2 The second type of sinkholes



(a) The first type of sea wall cross section



(b) The second type of sea wall cross section

Fig. 1 The schematic sketch of the two types of sea wall cross section

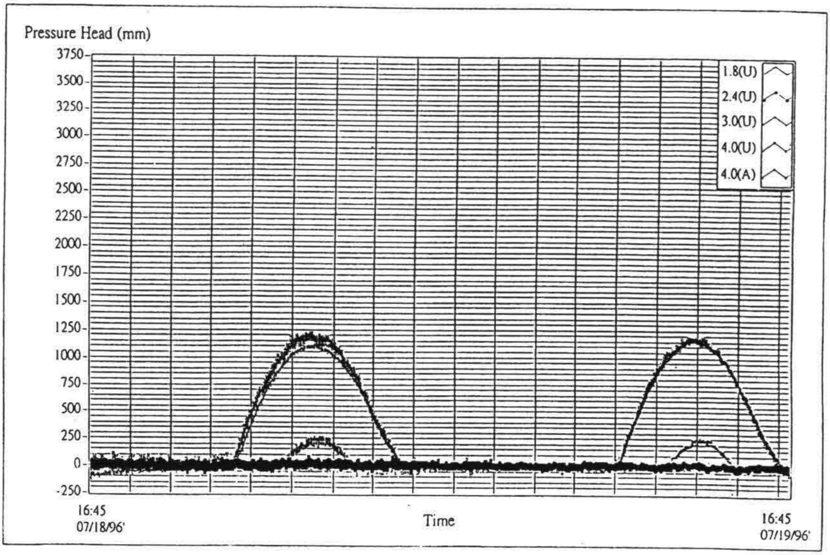


Fig. 2 Field data recorded between July 18th and 19th

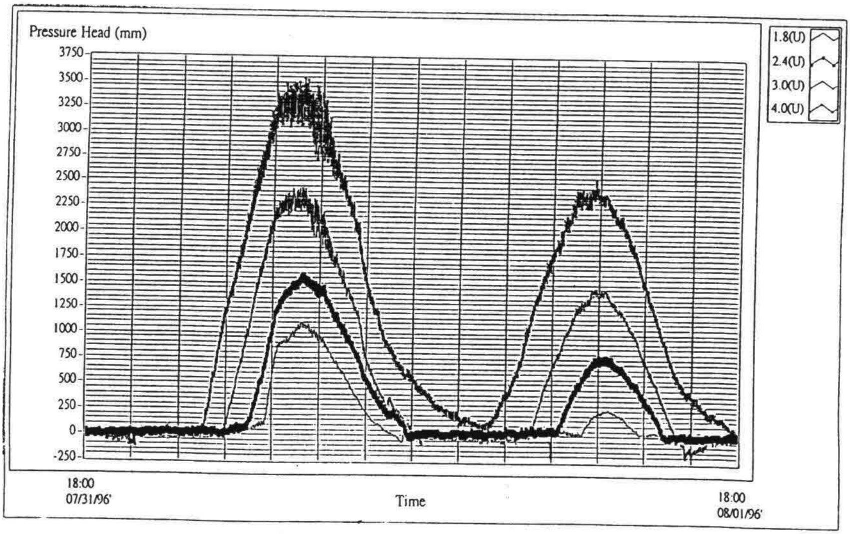


Fig. 3 Field data recorded during the attack of typhoon Herb

A Study on Tautly-moored Spar Buoy for Offshore Mariculture Cage

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Abstract

Due to the 200-mile Exclusive Economic Zone (EEZ), the environmental impact of Inland mariculture, such as land subsidence in western coastal plain of Taiwan and the increasing demand for sea food, open ocean mariculture is one of the major tasks of Taiwan fishery. However, typhoons invade Taiwan frequently in summer and offshore cages must sustain heavy waves of 10 meter high.

Four tautly-moored spar buoys are employed to hang an offshore cage, According to theoretical analysis and experiment, the wave height, wave period and water depth dominate the pitch motion in waves. A spar of 40~50 cm in diameter, 10~15 meter in length and 30~60m in water depth is suggested. The spar buoy and cage are quite stationary in waves. Further studies are necessary to understand whether the proposed system is suitable for open ocean mariculture or not.

Key words:tautly-moored spar buoy, wave, pitch angle, water depth.

1 Introduction

World fisheries will almost certainly not meet 21st century demands for seafood and other marine bioproducts. The present total of world fishery production is about 100 million metric tons, in which the aquaculture amounts to 9.8 million metric tons. An anticipated doubling of the world population in the coming century would lead one to anticipate a demand for at least 200 million metric tons of total world fish production within 50 to 70 years(Bardach, 1991). Recently Norway has become the major salmon export country in the world. However, fish farming has a short history in Norway, but the industry has developed extremely rapidly. At the beginning, the cages were anchored in a sheltered bay and very close to the shore. Due to the generally low water exchange rate, the fish suffered bad water quality. After several years at the same location, the production rate decreased and some of the fish farms obviously polluted the environment. The cages moved gradually offshore and to open ocean(Dahle, 1991). Offshore mariculture offers some specific advantages over inshore mariculture from both the production and environmental standpoints. Fluctuations in water temperature and salinity are more gradual and of lesser amplitude. Water quality and circulation are normally better than those at inshore locations. It follows that the disposal of organic waste would be improved at offshore site, thus minimizing local eutrophication and organic enrichment of the benthic sediment and reducing the potential for deleterious environmental impact. Otherwise, offshore mariculture may lower the degree of risk of disease transmission if the culture facility is positioned in an area of relatively low fish density(Donaldson, 1991).

Due to the 200-mile Exclusive Economic Zone(EEZ), the environmental impact of inland mariculture, such as land subsidence in western coastal plain of Taiwan, and the increasing demand for sea food, open ocean mariculture is one of the major tasks of Taiwan fishery. Unfortunately, there is hardly a site of sheltered sea area. Moreover, typhoons invade Taiwan frequently in summer and offshore cages must sustain heavy waves of 10 meter high. Although there are several designs of submerged cages which can survive in a hostile sea condition, these cages are sophisticated but not economical. Hence, it is necessary to develope a native offshore mariculture cage. A principle is that the more stationary the cage is in waves, the better it can resist the waves. A tautly-moored spar buoy may be quite stationary in waves, and then the cage fastened to the spar buoys may also be stationary in waves.

2 Theoetical analysis in response of tautly-moored spar buoy in waves

For simplicity, a theoretical analysis of tautly-moored spar buoy in waves is based on the following assumptions:

(1) The mooring cable is always taut, but the elongation of the cable is neglected.

(2) The diameter of the cable is so small that its fluid drag, buoyancy and gravitational forces can be neglected.

(3) The hydrodynamic coefficients of spar buoy are independent of particle velocities of waves.

(4) The spar buoy is submerged in water and the wind force is neglected.

(5) The center of gravity and buoyancy center of the spar buoy coincide.

(6)The tension of cable is quite large that the cable and the buoy are considered as a rigid body which takes the anchor as a turning point.

The coordinate system is as shown in Fig. 1. According to Newton's 2nd law of motion, the equations of motion for

the spar buoy are the following:

Surge $$m\ddot{x} = \sum_{j=1}^{5} F_{xj} \tag{1}$$

heave $m\ddot{z} = \sum_{j=1}^{5} F_{zj}$ (2)

pitch $I\ddot{\theta} = \sum_{j=1}^{5} M_j$ (3)

in which m is the mass of the buoy, F the force, M the moment and I the moment of inertial and equal to

$$I = \frac{1}{3}mL^2 + ml_0\left(l_0 + L\right)$$

in which l_0 length of mooring line

L length of buoy

The forces and moments are the following:

(1) Buoyant forces and moments(j=1)

$$F_{x1} = 0 \tag{4}$$

$$F_{z1} = \rho_w Vg \tag{5}$$

$$M_1 = -F_{z1}(l_0 + \frac{L}{2})\sin\theta \tag{6}$$

in which V is the volume of buoy and θ the pitch angle.

(2) Gravity(j=2)

$$F_{x2} = 0 \tag{7}$$

$$F_{z2} = -W_0 \tag{8}$$

$$M_2 = -F_{z2}(l_0 + \frac{L}{2})\sin\theta \tag{9}$$

in which W_0 is the weight of spar buoy.

(3) Viscous forces and moment(j=3)

Following Morison equation and Huang(1990), the viscous forces and moment are as follows:

$$F_{x3} = \int_{a}^{c} \frac{1}{2}\rho_w C_{dx} D(U - \dot{X})\left|U - \dot{X}\right| dz \tag{10}$$

$$F_{z3} = \frac{1}{2}\rho_w C_{dz} A'\left(\overline{W} - \dot{Z}\right)\left|\overline{W} - \dot{Z}\right| \tag{11}$$

$$M_3 = \int_{ra}^{rc} \frac{1}{2}\rho_w C_{dx} D(U - \dot{X})|U - \dot{X}|(h+z)\sec\theta\cos\theta dz$$
$$+F_{z3}\left(l_0 + \frac{L}{2}\right)\sin\theta \tag{12}$$

in which U is the water particle velocity in X direction, W the water particle velocity in Z direction, D the diameter of spar buoy, A' the projected area of spar buoy in Z direction and ra rc are the integral limits of spar buoy in Z direction.

C_{dx} is the drag force coefficient in surge and C_{dz} the drag force coefficient in heave.

And,

$$\overline{W} = \frac{1}{L}\int_{ra}^{rc} W_{dz}$$

(4) Added mass forces and moment(j=4)

Following Morison equation and Huang(1990), the added mass forces and moment are as follows:

$$F_{x4} = \int_{ra}^{rc} \rho_w (C_{ax} + 1)\left(\frac{\pi D^2}{4}\right)\dot{U} dz - \rho_w (C_{ax} + 1)V\ddot{X} \tag{13}$$

$$F_{z4} = \rho_w (C_{az} + 1)V\overline{\dot{W}} - \rho_w (C_{az} + 1)V\ddot{Z} \tag{14}$$

$$M_4 = \int_{ra}^{rc} \rho_w (C_{ax} + 1)\left(\frac{\pi D^2}{4}\right)\dot{U}(h+z)dz -$$
$$\rho_w (C_{ax} + 1)V\ddot{X}\left(l_0 + \frac{L}{2}\right)\cos\theta + F_{z4}\left(l_0 + \frac{L}{2}\right)\sin\theta \tag{15}$$

in which $\overline{\dot{W}} = \frac{1}{L}\int_{ra}^{rc} W_{dz}$

C_{ax} is the added mass coefficient in surge and C_{az} the added mass coeffient in heave.

(5) Tension(j=5)

The tension force of mooring line exerts at the lower end of spar buoy,as the following:

$$F_{x5} = -T_x \tag{16}$$

$$F_{z5} = -T_z \tag{17}$$

$$M_5 = 0 \tag{18}$$

Substituting equations(4)-(18) into Equations(1)-(3), one gets the following equations:

Surge

$$\left[m+\rho_w(C_{ax}+1)V\right]\ddot{X}=\sum_{j=1}^{3}F_{xj}+\int_{ra}^{rc}\rho_w(C_{ax}+1)A'\dot{U}dz-T_x \tag{19}$$

Heave

$$\left[m+\rho_w(C_{ax}+1)V\right]\ddot{Z}=\sum_{j=1}^{3}F_{zj}+\rho_w(C_{az}+1)V\overline{\dot{W}}-T_z \tag{20}$$

Pitch

$$\begin{aligned}&I\ddot{\theta}+\rho_w(C_{ax}+1)V\ddot{X}\left(l_0+\frac{L}{2}\right)\cos\theta=\sum_{j=1}^{3}M_j+\int_{ra}^{rc}\rho_w(Cax+1)A'\dot{U}(h+z)dz\\&+F_{z4}\left(l_0+\frac{L}{2}\right)\sin\theta\end{aligned} \tag{21}$$

From field experimental experience, if the length and the buoyancy of the spar buoy are enough, the pitching angle is kept very small. Then the hydrodynamic forces in z-direction and the influence by pitch on the hydrodynamic forces in x-direction may be neglected. Further, $(U-\dot{X})|U-\dot{X}|$ is simplified to be $(U-\dot{X})|U|$, which is then linearized. Then eq. (21) becomes

$$A\ddot{\theta}+B\dot{\theta}+C\theta=F\cos\sigma t+E\sin\sigma t \tag{22}$$

in which

$$A=\left(\frac{\rho_s}{\rho_w}+C_{ax}+1\right)\frac{\pi D^2L}{4}\left[\frac{L^2}{3}+l_0(l_0+L)\right]$$

$$\begin{aligned}B=&\frac{C_{dx}DHg}{2\pi\sigma\cosh k(l_0+L)}\{[(l_0+L)^2\sinh k(l_0+L)C-\frac{2(l_0+L)}{k}\cosh k(l_0+L)+\\&\frac{2}{k^2}\sinh k(l_0+L)]-[l_0^2\sinh kl_0-\frac{2l_0}{k}\cosh kl_0+\frac{2}{k^2}\sinh kl_0]\}\end{aligned}$$

$$C=\left(1-\frac{\rho_s}{\rho_w}\right)\frac{\pi D^2L}{4}g\left(l_0+\frac{L}{2}\right)$$

$$\begin{aligned}F=&\frac{C_{dx}DH^2gk}{6\pi\sinh 2k(l_0+L)}\{L(2l_0+L)+\frac{1}{k}[(l_0+L)\sinh 2k(l_0+L)-l_0\sinh 2kl_0]\\&-\frac{1}{2k^2}[\cosh 2k(l_0+L)-\cosh 2kl_0]\}\end{aligned}$$

$$E = -\frac{(C_{ax}+1)\pi D^2 Hg}{\cosh k(l_0+L)}\{[k(l_0+L)\sinh k(l_0+L) - kl_0 \sinh kl_0]$$
$$-[\cosh k(l_0+L) - \cosh kl_0]\}$$

H is wave height, σ is wave frequency and k is wave number.

Eq.(22)is a second order ordenary differential equation. The solution consists of homogeneous and particular ones. The former is a transient solution and vanishes when time elapses. Therefore, only particular solution is left, i.e.

$$\theta(t) \rightarrow \theta_P(t) = \theta_{max}\cos(\sigma t - \beta) \tag{23}$$

$$\theta_{max} = \frac{\sqrt{F^2+E^2}}{\sqrt{(A\sigma^2 - C)^2 + \sigma^2 B^2}} \tag{24}$$

$$\beta = \tan^{-1}\left(\frac{E(-A\sigma^2+C)+F(\sigma B)}{F(-A\sigma^2+C)-E(\sigma B)}\right) \tag{25}$$

After eq.(24), a lower wave height H, a larger wave frequency £m, a deeper water depth h, a shorter spar buoy length L, a larger specific density £ls and a smaller diameter D will reduce the pitch angle. As above-mentioned assumption (6), the tension of cable is quite large, i.e. the net buoyancy must be large enough. This means that the length and diameter should be large enough and the specific density must be small. Otherwise, for hanging cage, which exerts drag force due to currents, the length and diameter of the spar should also be large. However, the last three parameters, i.e. L, £ls and D, are very insensitive. Hence, these parameters can be chosen according to practical needs.

3 Field experiment

Because we were in a hurry to know the result, a field test was carried out at a coastal site near Keelung before the theoretical analysis. The spar buoy, which is 300 cm in length and 20cm in diameter, is made of a stainless steel frame and several buoyant cylinders. On top of the spar buoy, a cylindrical case is fixed, in which a Dual-axis Inclinometer(SSY0090) and a recorder are installed. The inclinometer can measure the inclinations in the two horizontal axes i.e, θ_x and θ_y. The principle inclination angle θ can be calculated by the following equation:

$$\theta = \sin^{-1}\sqrt{\sin^2\theta_x + \sin^2\theta_y}$$

At the bottom of the buoy, there is a room, in which 5 iron discs(5.5 kg each) can be put as a ballast weight. A schema of the model is shown as in Fig. 2. A waverider was imployed to measure the wave. The experiment was executed by a fishing boat, two divers and two assistants. The field experiment lasted about 2 hours and the wave record is shown in Table 1. Two spar buoys were tested and $5m \times 3m$ fishing net was hanged on the buoys in some cases. The results are summarized in the following 6 cases as shown in Table 2.

Table 1 Wave data

Keelung 1994/12/22

Time	Hmax	Tmax	H1/10	T1/10	H1/3	T1/3	Hmean	Tmean
11:0	1.76	8.64	1.38	8.83	1.13	8.25	.70	5.83
11:20	1.61	8.32	1.33	7.97	1.08	7.79	.65	5.35
11:40	1.78	7.35	1.54	7.78	1.28	8.04	.80	5.95
12:0	1.89	8.68	1.59	7.98	1.35	8.17	.86	6.05
12:20	2.07	9.22	1.69	8.94	1.37	8.97	.84	6.37
12:40	2.33	9.23	1.68	8.72	1.34	8.50	.86	6.63
13:0	2.16	7.51	1.69	9.26	1.35	8.54	.83	6.26

Table 3 Field experiment data

No.	$H_{1/3}$ (m)	Hmax (m)	$T_{1/3}$ (sec)	number of iron disc	Distance between centers of gravitybuoyancy (cm)	net	θ_{max}
1.	1.08	1.61	7.79	5	57	no	5
2.	1.28	1.78	8.04	5	57	no	8
3.	1.35	1.89	8.17	5	57	yes	5
4.	1.37	2.07	8.97	5	57	yes	5
5.	1.34	2.33	8.50	3	50	no	5
6.	1.35	2.16	8.54	0	24	no	9

From the 6 cases, it is found that the ballast weight does not have significant influences on the pitch angle of a tautly-moored spar buoy. But the fishing net may reduce the angle. For case 5 and 6 in Table 2, the distances between centers of gravity and buoyancy are smaller. Therefore, these cases match the above-mentioned assumption (5) approximately. By applying eq.(24) and assuming Cax=1 and Cdx=1.5, £cmax is 5.7¢X for case 5 and 5.5¢X for case 6 ,in which water depth is 18m and £ls are 0.51 and 0.35 respectively. The result of case 5 match each each other, but not for case 6. However, the results of case 6 still belong to the same order of magnitude.

4 A tentative design of an offshore fish farming

According to theoretical analysis and field experiments, a spar buoy of 40~60 cm in diameter and 1000~1500 cm in length is suggested. A concrete block of 3-5 tons is used as an anchor, which can be reused. The deployment of the blocks is rather a hard work. The water depth is about 20-60 meters. The small buoys are to support the weight of the cage and spar buoys are to take the burden of drag force due to current. One cage is fastened to four spar buoys and each spar buoy is tied with a upper and lower ropes to the cage. Because the tautly-moored spar buoy is almost stationary in waves, the cage is also quite stable. This may be favorable for mariculture.

The current usually flows along the coastline. Then it is better to employ rectangular cages. The drag force by current is as follows:

$$F_d = \frac{1}{2} C_d \rho_w A V^2$$

in which F_d drag force in newtons

C_d drag coefficient

ρ_w the seawater density

V current speed in m/s

A projected area of a body to the approaching current velocity

The drag coefficient of a cylinder is about 0.8-1.3 depending on the Reynolds number. The drag coefficients of netting material was presented by Milne (1967). He demonstrated that the marine fouling increases the coefficients significantly. The formulae for the coefficient of drag on clean, unfouled nets are as follows:

For a knotted net

$$C_d = l + 3.77(d/l) + 9.37(d/l)^2$$

and for a knotless net

$$C_d = l + 2.73(d/l) + 3.12(d/l)^2$$

in which d diameter of the net material in inches

l length of one bar of the mesh in inches

An example to check the drag of current is shown as follows:

knotted net

diameter of net material d	0.06"
length of one bar l	0.4"
current speed V	0.5 m/s

dimension of net pen	length 12m, width 8m, height 8m
diameter of spar buoy	0.4m
length of spar buoy	10m
drag coeficient C_d	1.3

The drag force of one spar buoy

$$1.3 \times 1000 \times 10 \times 0.4 \times 0.5 \times 0.5 / 2 = 650(N.T.) \cong 66.3kg$$

The projected area of one smaller side of the net pen

$$(0.4 - 0.06)^2 / 0.4^2 = 0.72$$

$$1 - 0.72 = 0.28$$

$8 \times 8 \times 0.28 = 17.92$ (square meter)

The projected area of the net pen A

$17.92 \times 2 = 35.84$ (square meter)

The drag coefficient C_d

$$0.4 + 3.77 \times (0.06 / 0.4) + 9.37 \times (0.06 / 0.4)^2 \cong 1.176$$

The drag force of the net pen

$$1.176 \times 1000 \times 35.84 \times 0.5 \times 0.5 / 2 \cong 5268(N.T.) \cong 527kg$$

527kg is distributed in two spar buoy.

According to eq.(24) and assuming H=9m, T=10sec, h=30m, D=0.4m, L=10m and £ls=0.2. £cmax is 13¢X. This is acceptable.

5 Discussion and conclusion

Although in the theoretical analysis there are assumptions that are not realistic, i.e. that the spar buoy and the mooring line are like a rigid body, the conclusion that wave height, wave period and water depth dominate the pitch motion in waves for a tautly moored spar buoy is supported by the field experiment. A practical design of offshore mariculture farm is to install 2 rows of tautly-moored spar buoys along coast. The distance between them is about 20 meters. 4 spar buoys can serve for one cage. Because each intermediate spar buoy serves for two cages,each cage require a little more than 2 spar buoys in average.The dimension is suggested to be 40 cm in diameter and 1000~1500cm in length. The design wave height Hm is 9m , wave period is 10sec, eater depth is 30m and current speed is 1 knot.If the wave is bigger, the water depth must be enlarged. The spar buoy and cage are quite stationary in waves. Whether this system is suitable for open ocean mariculture in Taiwan or not, further field experiments are needed.

6 Acknowledgement

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Wave Impact Loading and Dynamic Response of Vertically Faced Breakwaters - European Research Project MAST III/PROVERBS -

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Abstract

The paper attempts (i) to briefly outline the main objectives, issues and motivations as well as the organisation and participation structure of PROVERBS, a European research project within the Marine Science of Technology Programme (MAST III) on probabilistic design of vertical breakwaters and (ii) to present some selected findings of this project related to the wave impact loading and dynamic response of caisson breakwaters.

1 Introduction

Due to the growing pressure exerted by human activities and needs in the industrial and amenity sector within the coastal zone, interest in protective structures against the hydrodynamic actions of the sea is expected to increase. A further important reason supporting this expectation is the increase of the strength and frequency of storm surges observed in the last decades.

On the other hand, the construction of coastal structures is still essentially based on empirical design methods, as well as on trial and error approaches, thus making any optimisation almost impossible. The latter statement is particularly supported by the fact that most of the catastrophic failures experienced by coastal structures could not be predicted at the design stage and cannot yet be satisfactorily explained by present design methods and analyses. In fact, most of the failure modes which have been identified to date are associated with the dynamic nature of the wave loads and the highly transient phenomena involved, suggesting an urgent need for the development of integrated rational design approaches based on an increased understanding of the hydrodynamic, geotechnical and structural processes involved in the wave-structure-foundation interaction.

With this background an extensive research programme, which also includes coastal structures, has been initiated by the European Union (EU) within the Marine Science and Technology (MAST), and one of the research projects within this phase deals with "Probabilistic Design Tools for Vertical Breakwaters (PROVERBS)". In the following a short presentation of PROVERBS is given, followed by a brief outline of some selected key findings and their practical importance.

2 Brief Presentation of PROVERBS

2.1 Main Objectives

PROVERBS is a research project involving 23 institutes from 8 European countries and different disciplines (fluid mechanics, applied mathematics, soil mechanics, structural dynamics, hydraulic and coastal engineering).

The main objective of PROVERBS is to develop and implement new rational probability based methods for the design of monolithic coastal structures and breakwaters subject to sea wave attacks, and then demonstrate their advantages as compared to existing conventional design methods. This will be achieved in PROVERBS by enhancing the knowledge in various aspects associated with hydrodynamic, geotechnical and structural processes involved in the wave-structure-foundation interactions and in the associated failure mechanisms. In addition, the results gained in further related MAST-Projects and national research projects will also be linked together with the new knowledge generated in PROVERBS and implemented for the development of the probabilistic design tools (Fig. 1).

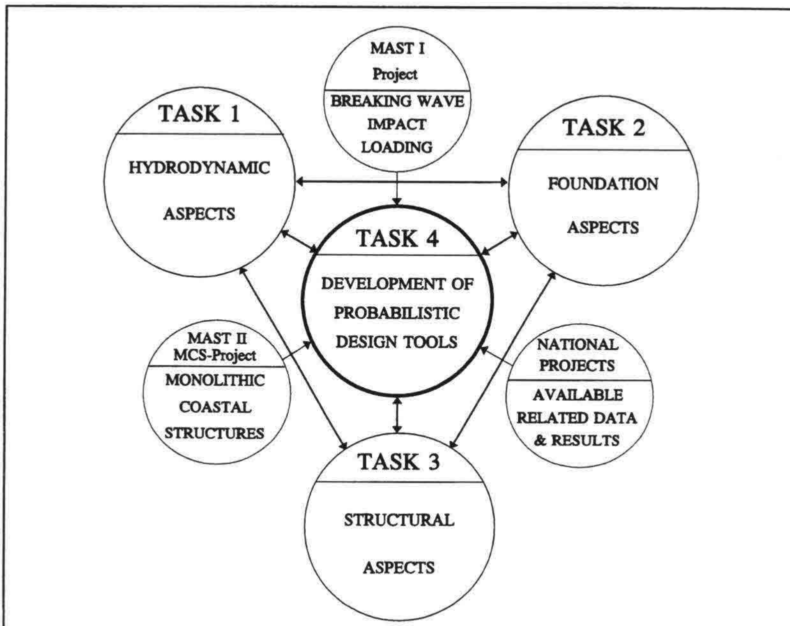


Fig. 1: Implementation of existing and new knowledge in probabilistic based design framework (PROVERBS)

These new design tools are intended to supplement/replace the existing conventional design approach and to build a solid scientific basis for any future authoritative European design guidelines and codes. Some of the monolithic structures/breakwaters addressed in PROVERBS are schematically shown in Fig. 2.

2.2 General Motivations

The major reasons why monolithic structures and why probabilistic design methods have been selected as candidate research topics for a large European project are outlined below.

2.2.1 Motivations for Monolithic Coastal Structures / Breakwaters

- *Catastrophic failures*: Numerous severe and catastrophic failures were experienced by vertical breakwaters in the 1930s. It should be stressed that major failures may cost 2-3 times more to re-build than the original construction costs. As a consequence, the vertical breakwater type was almost abandoned - except in some few countries - in favour of the rubble mound breakwater type (*Oumeraci, 1994*). In Japan for instance, about 7 failures per year are experienced by vertical breakwaters. After a series of catastrophic failures experienced by large rubble mound breakwaters at the end of the 1970s and the beginning of the 1980s, a number of actions were started to promote the revival of vertical breakwaters and the developments of new breakwater concepts (*Oumeraci et al, 1991*). In this respect extensive research efforts at interdisciplinary and multinational level were urgently required.

- *Need for breakwaters at greater depth*: To suit the increasing draught of large vessels, breakwaters should be founded in increasingly deeper water, thus making the cost of such structures more prohibitive. Construction costs of 100 to 350 million ECU/km breakwater are not seldom. In this respect, a type of structure is needed which represents a better alternative not only in terms of technical performance and total costs, but also in terms of standardization, quality control, environmental aspects, construction time and maintenance. Moreover, more focus than in the past must be put on the optimisation of the design.

- *Need for environmently friendly structures*: Monolithic structures can easily be given any shape, perforations and any further constructional features to reduce the impact on the environment. Moreover, less material and less energy for material transportation than for further traditional breakwater types is required for construction. In fact, most of the material involved in a caisson breakwater is sand - dredged from deeper sea, thus minimizing the energy required for transportation and maximising the conservation of scarce construction material.

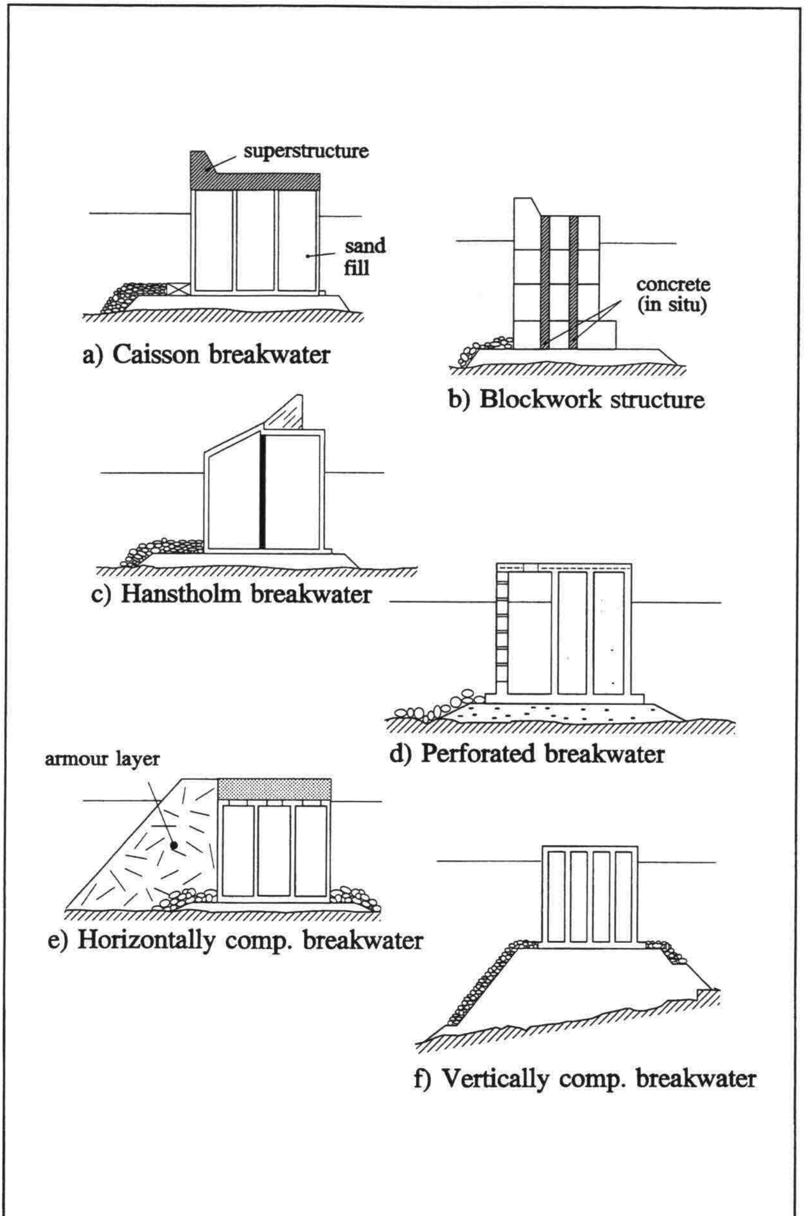


Fig. 2: Main types of monolithic structures/breakwaters

- *Need for multi-purpose structures*: Due to the inherent considerable public investments, it is expected that the objective of coastal structures will not solely be limited to the damping of wave action. Taking the opportunity of such expensive structures, facilities for amenity and wave power extraction might also be integrated in the structure (cost sharing, acceptance by society etc.). In this respect, caisson structures are suited for this purpose, because of their flexibility to adapt to any requirements related to their shape, size etc.. Although not yet based on rational design methods, the caisson type structures have already demonstrated their capability because they can easily be adapted to meet also:
  - further purposes like amenity, wave power extraction and further industrial needs;
  - environmental requirements by providing a suitable shape and further interesting features to reduce wave reflection, to increase the water exchange between open sea and protected area, to minimize the disturbance of the ground and to fit into the maritime scenery.

  Examples from Japan (*Tanimoto et al., 1994*), Monaco (*Bouchet et al., 1994*) and Korea (*Lee et al., 1994*) have already shown that the potential of adapting caissons type structures to meet any requirement of technical, social and ecological nature is higher than for any other traditional type of structure. This however, requires a high level of knowledge and technology. PROVERBS is expected to contribute to bringing the European maritime construction industry in a world leading position in this field.

- *Potential large-scale application for sea-walls*: Because of the competitiveness of caisson structures in terms of technical performance, total costs, environment, quality control, construction time and standardization, it is believed that there is also a large potential for their use as sea walls to respond to the increase of storminess and sea level rise. This will help to react more rapidly and better protect the coastal zones of high economic, social and environmental values.

2.2.2 Motivations for Probabilistic Design Methods

- *Need for more and better optimisation*: Breakwaters and coastal structures represent considerable public investments (in the range of 1 billion US$ for a 5 km long breakwater in deep water) which encounter less and less acceptance by government and local authorities, due to the decreasing availability of fundings for this kind of large projects and the increasing awareness of environmental impacts by governments and society. Beside the need to diversify the use of such structures (see multi-purpose use as described above), there is an urgent need to use more rational methods for design and more sophisticated tools for the optimisation of such structures. It is obvious that such methods can only be developed in an integrated manner, requiring a multi-national framework and a multi-disciplinary research strategy.

- *Complexity of physical problems involved*: The results of previous MAST-projects have highlighted the complexity and the integrated nature of the problems related to breakwater stability and design, including the considerable importance of the three-dimensional and stochastic nature of the processes involved in the wave-structure-foundation interactions (wave loading and dynamic response) as well as the large number of possible failure mechanisms and their complex interaction. This necessarily prescribes the use of probability-based analysis methods as the sole alternative for the design. In fact, this is the only alternative which may provide a systematic and comprehensive framework not only for optimisation procedures but also for the application of engineering judgement.

- *Stimulating aspects*: Since the prospective probability based design methods will essentially be based on the feedback from prototype experience, the results are expected to stimulate a rapid and continuous feedback between researchers and practioners, thus enhancing the worldwide applicability of these tools. This will also actively stimulate the collection of more and better data, because it is the essence of probability based tools to use more and better information.

2.3 Issues/Tasks Addressed by PROVERBS

As already illustrated by Fig. 1, four main issues are addressed by PROVERBS: hydrodynamic, foundation, structural and probabilistic design aspects. Each of these aspects is dealt with in a sub-project called "Task".

- *Task 1: Hydrodynamic Aspects*: Hydrodynamic inputs will be provided by focusing on wave loadings, including associated uncertainties and statistical distributions required to implement the probabilistic design tools in Task 4. Prototype measurements and 3D-hydraulic model tests will form the major part of this work, supplemented by further physical and numerical modelling.

- *Task 2: Foundation Aspects*: Beside the development of new knowledge related to failure mechanisms and dynamic soil properties, information will be generated on probability, partial safety factors and uncertainties for soil parameters and models, making them readily applicable for the implementation of probabilistic design tools in Task 4. Analysis, numerical modelling and laboratory tests will build the major means for the investigations, supplemented by centrifuge modelling and prototype measurements.

- *Task 3: Structural Aspects*: This task will provide the methods to assess the structural strength of breakwater walls under pulsating and extreme impact loading, as well as their durability. Uncertainties in the loading and resistance parameters will be accounted for as needed for the implementation of the probabilistic design tools. Finite element analysis will principally be used for the investigations.

- *Task 4: Probabilistic Design Tools*: In a first preparatory phase a probabilistic framework will be developed by linking together all above aspects by describing the failure modes, by establishing the fault trees, by evaluating the associated uncertainties and developing a reliability design philosophy. In a second phase, probabilistic design tools will be developed. Based on reliability calculations at Level II and III, on the analysis of case studies and involvement of practitioners, a method to assess the overall reliability of vertical breakwaters will be developed and applied to a set of selected representative structures.

2.4 Organisation and Participation Structure of PROVERBS

The 23 partners from 8 European countries involved in PROVERBS are given in Tab. 1.

The organisation structure showing the Coordinator, the Management Committee Meeting and the Task Leaders is given in Fig. 3.

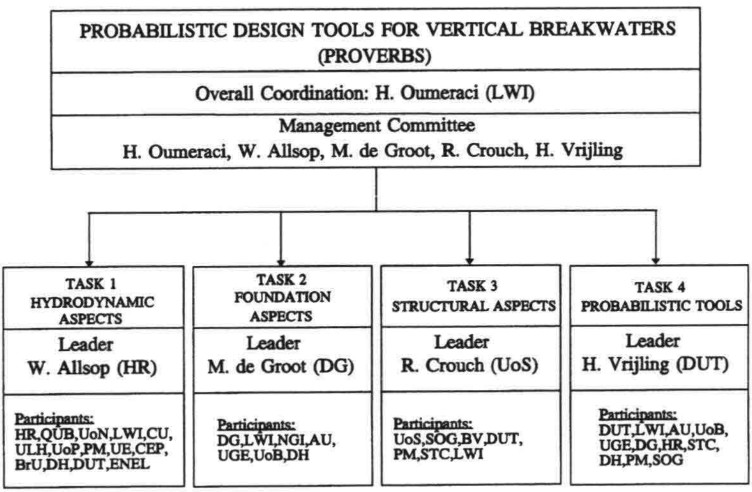


Fig. 3: Organisation structure of PROVERBS

No	Code	Name and location	Cont. person	Nat.	Role
01	LWI	Leichtweiß-Institut (TU Braunschweig)	H. Oumeraci	DE	CO[1)]
02	HR	Hydraulic Research Wallingford	W. Allsop	GB	PA[1)]
03	DG	Delft Geotechnics	M. de Groot	NL	PA[1)]
04	UoS	University of Sheffield	R. Crouch	GB	PA[1)]
05	DUT	Delft University of Technology	H. Vrijling	NL	PA[1)]
06	AU	Aalborg University	H. Burcharth	DK	AP
07	BrU	University of Bristol	H. Peregrine	GB	AP
08	CEP	Centro de Estudios de Puerto y Costas	B. Madrigal	ES	AP
09	DH	Delft Hydraulics	J. van der Meer	NL	AP
10	PM	Politecnico di Milano	L. Franco	IT	AP
11	UoP	University of Plymouth	P. Hewson	GB	AP
12	UoB	University of Bologna	A. Lamberti	IT	AP
13	UGE	Universität Gesamthochschule Essen	W. Richwien	DE	AP
14	ULH	University of Le Havre	M. Bélorgey	FR	AP
15	SOG	SOGREAH	A. Martinez	FR	AP
16	NGI	Norwegian Geotechnical Institute	K. Andersen	NO	AP
17	UE	University of Edinburgh	B. Easson	GB	AP
18	CU	Cantabria University	I. Losada	ES	AP
19	QuB	Queens University of Belfast	G. Müller	GB	AP
20	UoN	University of Naples	E. Benassai	IT	AP
21	BV	Bureau Veritas	J. Isnard	FR	AP
22	STC	Service Technique Central Compiègne	J.-B. Kovarik	FR	AP
23	ENEL	ENEL Sozieta per Azioni	M. di Gerloni	IT	AP

CO = Coordinator; PA = Partner; AP = Associated Partner

Tab. 1: Participation structure of PROVERBS

The duration of the project is 36 months starting at 1st February 1996 and running to end of January 1999.

1) Full addresses of CO and PA are given in Annex A.

3 Selected Key Findings and Their Practical Importance

Some of the selected results which have been achieved in PROVERBS, including some findings of the previous MCS (Monolithic Coastal Structures) project which have been completed within PROVERBS are briefly outlined below.

3.1 Parameter Map for Structure, Wave and Loading Conditions

The prediction of breaking waves and associated wave loads is still not amenable to a theoretical treatment. Extensive laboratory testing at HR Wallingford (*Allsop et al., 1996*) and further institutes has led to the parameter map shown in Fig. 4.

The parameter map allows us to identify the regions of structure, depth and wave parameters for which conventional prediction formulae for wave loading may safely be used, as well as those regions where new prediction methods are required. The parameter map has also allowed to derive a structure classification based on wave conditions and loading. This map does not only build the basis for any further research on wave loading within and beyond MAST, but also strongly help avoiding a misuse of any prediction formulae which might result in failures costing billions of ECU. In fact, the new development and further refinements of prediction formulae which are in progress in PROVERBS will provide an unique set of reliable wave loading formulae for the full range of structure parameters, depth and wave conditions, including the assessment of hydraulic performance of the structures (reflection, transmission, overtopping). Further details on the parameter map are given by *Allsop et al. (1996).*

3.2 Classification of Wave Loading

Depending on the purpose for which they will be used three types of wave loading have been clearly specified (Fig. 5):

This wave load classification supplements the parameter map in Fig. 4 by specifying the wave loading in more details, so that improved wave load criteria can be properly selected according to the type of failure mode under study (single failure or progressive failure). Further details on wave load classification are given by *Oumeraci and Kortenhaus (1997).*

3.3 Prediction Formulae for Breaking Wave Impact Loads

Based on solitary wave theory, impulse-momentum considerations, extensive laboratory tests, sophisticated measuring techniques (PIV) and large-scale model tests, design formulae have been derived for the prediction of the relative impact forces $F^*_{h,max} = F_{h,max}/(\rho \cdot g \cdot H_b^2)$ as a function of the relative rise time $t^*_r = t_r/\sqrt{(d_b/g)}$ (Fig. 6).

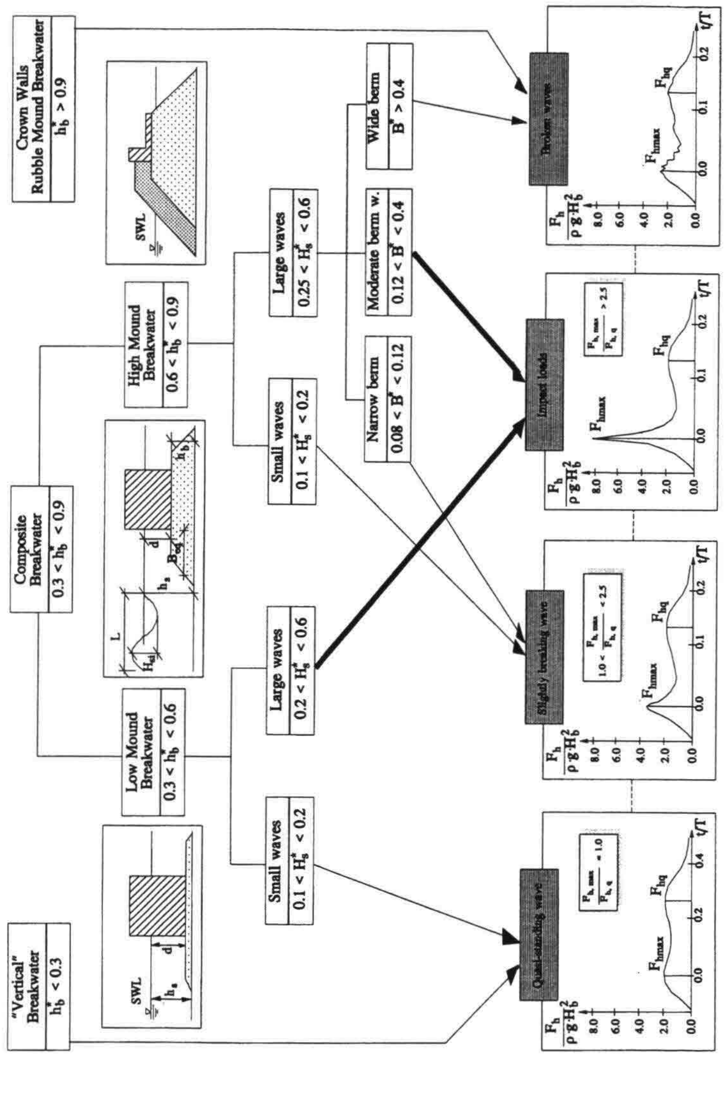


Fig. 4: Parameter map for wave loading of monolithic structures

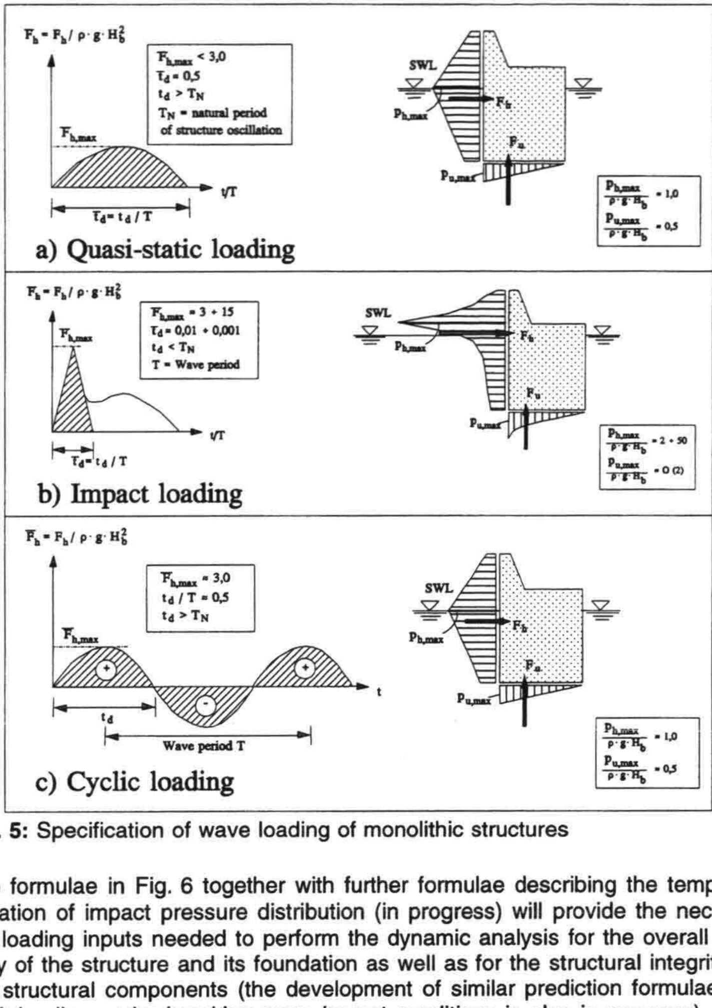
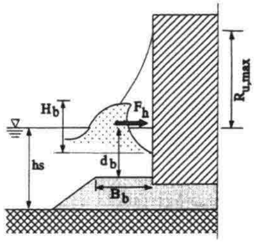


Fig. 5: Specification of wave loading of monolithic structures

The formulae in Fig. 6 together with further formulae describing the temporal variation of impact pressure distribution (in progress) will provide the necessary loading inputs needed to perform the dynamic analysis for the overall stability of the structure and its foundation as well as for the structural integrity of the structural components (the development of similar prediction formulae for uplift loading under breaking wave impact conditions is also in progress). The achieved and prospective results represent an important step towards the era of probabilistic and dynamic design approaches instead of the existing conventional static design approaches which can neither explain the failure modes observed in the prototype, nor allow any design optimisation. Overall, this will result in much safer structures without resulting in more costly design. Further details on impact loading related to horizontal wave impact loads are given by *Oumeraci and Kortenhaus (1997)* and to uplift loads under impact conditions by *Kortenhaus and Oumeraci (1997)* and *Geotechnical Group (1997)*.

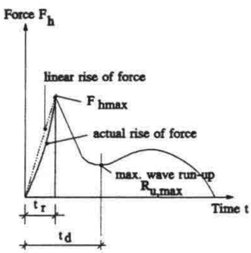
WAVE IMPACT FORCE



$$F_h^* = 2.24 \cdot \left(t_r^*\right)^{-1} \qquad (1)$$

with $F^*_{h,max} = F_{h,max}/(\rho \cdot g \cdot H_b^2)$

$t^*_r = t_r/\sqrt{(d_b/g)}$



Impact Duration:

$$t_d = t_r + 0.35 \cdot \left(1 - \exp(-20 \cdot t_r)\right) \qquad (2)$$

F_{hmax} in Eq. (1) is assessed from statistical distribution of impact force (log-WEIBULL distribution):

$$P(\hat{F}^*_{hmax}) = 1 - \exp\left\{-\left[\gamma(\ln - \beta)\right]^{\alpha}\right\} \qquad (3)$$

with $\alpha = 2.26$, $\beta = 0.74$ and $\gamma = 1.19$.

Fig. 6: Prediction formulae for breaking wave impact forces

3.4 Effect of Breaking Wave Impacts on Structure Response

The results of extensive model testing in a small and a large wave flume have shown that breaking wave impacts may generate oscillating and permanent displacements (Fig. 7).

These results have led to the development of numerical models for the oscillating motions as well as for the permanent displacements of the structure which both affect the stability of the structure and its foundation. The developed numerical models include both simple engineering models (*Oumeraci and Kortenhaus, 1994*; *Kortenhaus and Oumeraci, 1995*) as well as more sophisticated research models for structure foundation interaction (*Lengnick and Abdel-Rahman, 1994*; *De Groot, 1997*).

3.4.1 Engineering Models for Dynamic Response

Simple engineering models have been developed and calibrated by large-scale model tests to simulate both oscillatory motions and permanent displacements of a breakwater subject to impact loading induced by breaking waves. The models can also be used to reproduce the cumulative effect of repetitive impacts as shown in Fig. 8.

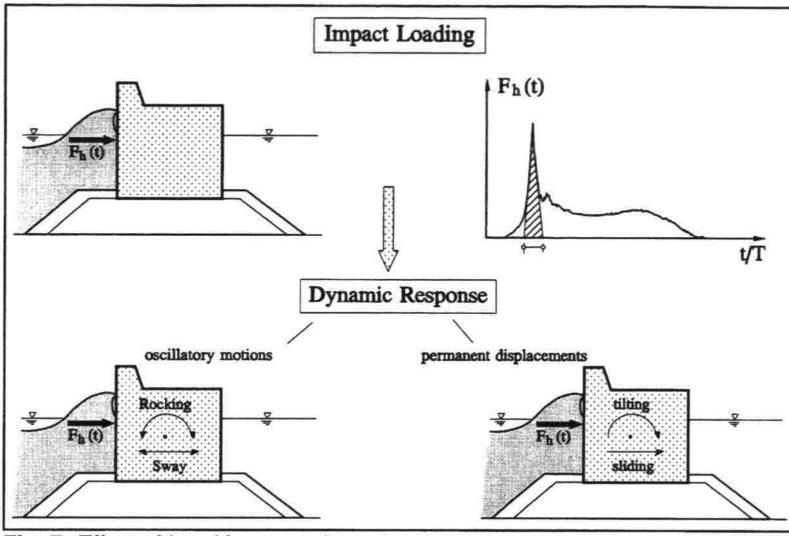


Fig. 7: Effect of breaking wave impacts on structure response

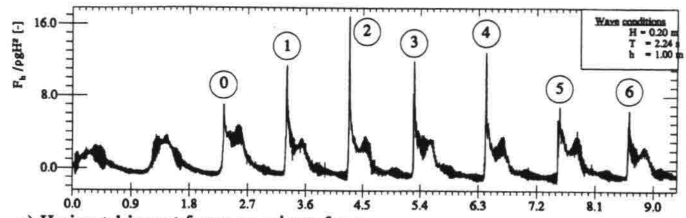
Both models can be used together to study the stepwise failures which were neglected in the past. In fact, the displacements induced by a single impact may be too small and non relevant for the stability, but the cumulative effect resulting from repeated impacts may lead to the collapse of the structure. These models have already been successfully applied to reproduce some Japanese prototype failures which otherwise could not be explained by existing standard formulae (*Oumeraci et al., 1995*). This results in safer design over the lifetime of the structure.

3.4.2 Research Models for Structure-Foundation-Interaction

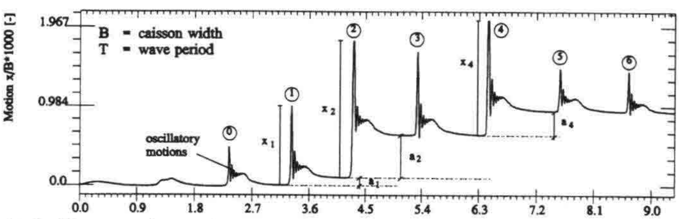
Some sophisticated finite element models have also been developed for the simulation of the dynamic response of the foundation when the structure is subject to impact loading.

These models are used as research tools to improve the physical understanding of the processes leading to stepwise failure. The example in Fig. 9 illustrates for instance that severe impact may induce plastic deformations of the foundation at the heel.

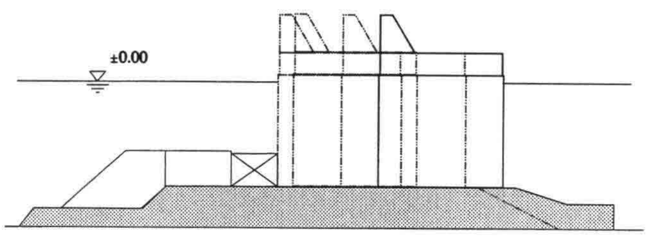
These and further similar results have highlighted the vital importance of the hidden foundation problems and the associated stepwise failures which were neglected in the past because coastal engineers and researchers 'did not feel competent to examine soil dynamics problems involved in breakwaters'. In particular the sliding failure which has been considered worldwide as the domi-



a) Horizontal impact forces on caisson front t/T [-]



b) Oscillatory motions and permanent displacements t/T [-]



c) Cumulative displacements of vertical breakwaters

Fig. 8: Cumulative effect of repetitive impacts on breakwater stability

nating failure mode is found to be in fact a result of geotechnical failure modes underlying the commonly observed overall failure modes like sliding and overturning. Also the seaward tilting mode by which numerous breakwaters dramatically failed in the past has now been identified as a geotechnical failure mode.

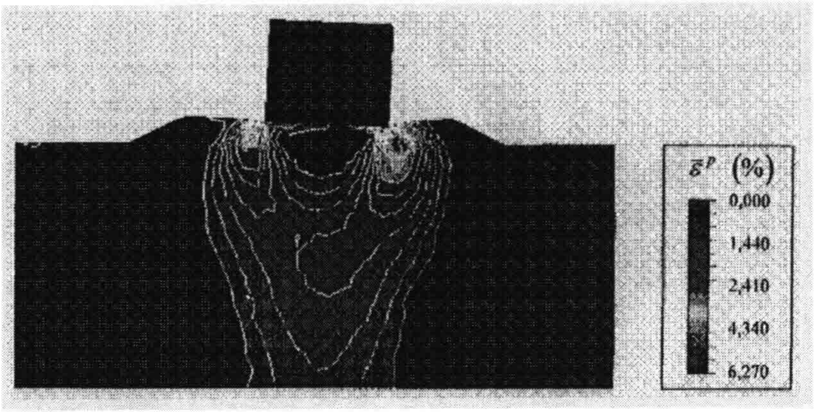


Fig. 9: Plastic deformation of foundation at the heel (*Lengnick et al., 1994*)

3.5 Scaling of Wave Impact Loads

Mathematical studies and laboratory testing have shown that even a very small fraction of air in water can dramatically reduce the impact pressure and increase the impact duration. This and the scaling problems illustrated by Fig. 10 point out towards the necessity of devoting more effort to estimate the volume fraction on entrained/entrapped air during impact. Moreover, comparative impact tests using fresh and sea water have shown that higher impact pressures are expected from fresh water tests than from similar sea water tests due to smaller bubble sizes and higher aeration levels in sea water. These efforts has led to the development of a new instrumentation to measure air content which is deployed in the laboratory and in the field (Fig. 10).

The results of the ongoing theoretical and experimental studies, together with the field measurements provide both researchers and engineers with the necessary tools to reliably interpret and transfer the results from hydraulic model tests to prototype conditions. Overall, this will result in considerable savings because Froude scaling which is still commonly used is too conservative. Further details are given by *Oumeraci and Hewson (1997)*, *Peregrine (1996)* and *Walkden et al. (1996)*.

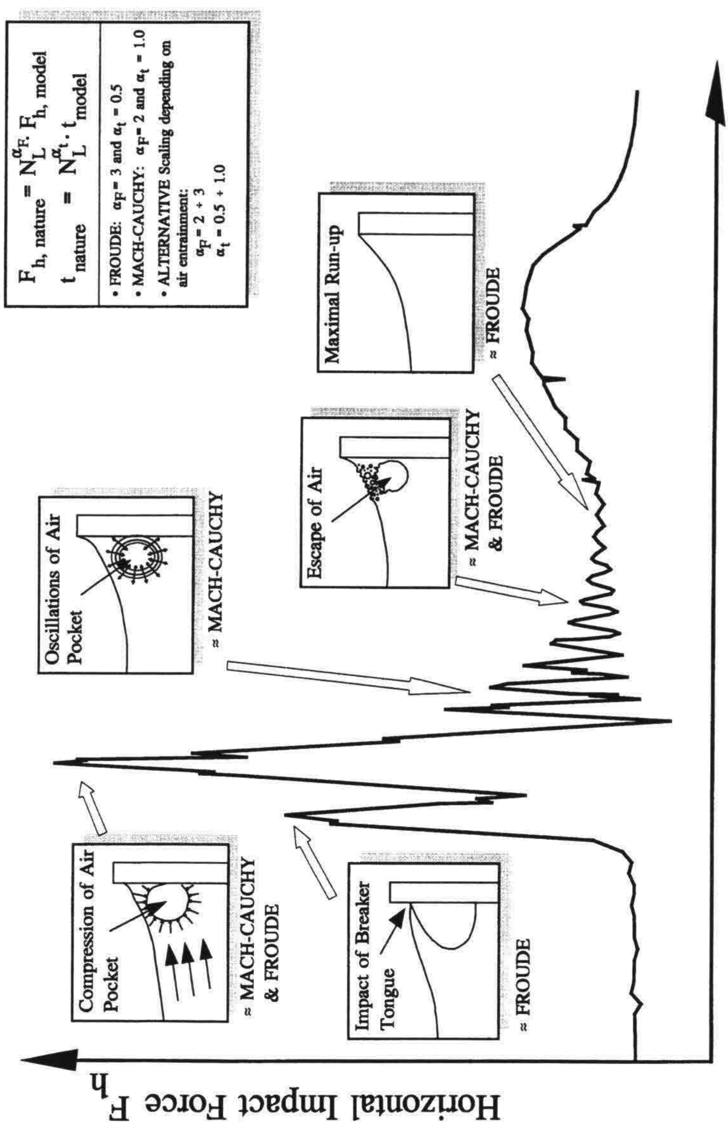


Fig. 10: Relevant scaling laws for the various features and processes of impact loading

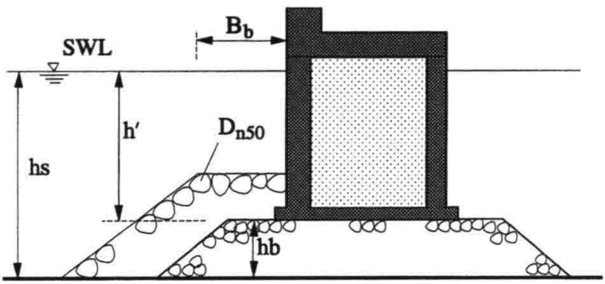
3.6 Toe Berm Stability

Comprehensive 2D-experiments resulted in a new stability formula to calculate the required average stone size D_{n50} as a function of the significant wave height H_s, water depth conditions and damage level (Fig. 11).

TOE BERM STABILITY

<u>Stability Number:</u>

$$\frac{H_s}{\Delta \cdot D_{n50}} = \left(5.8\,\frac{h'}{h_s} - 0.6\right) N_o^{0.19}$$



valid for: $0.50 < h'/h_s < 0.80$
$7.5 < h'/D_{n50} < 17.5$
$0.30 < B_b/h_s < 0.55$

where:

- Δ = $(\rho_S/\rho_W - 1)$; relative stone density
- D_{n50} = average stone size
- H_s = significant wave height
- N_o = number of displaced stones from a strip of a length $l = D_{n50}$ along the breakwater
  - $<$ 0.5: no damage
  - $\leq$ 2.0: admissible damage
  - $\geq$ 5.0: inadmissible damage

Fig. 11: Toe berm stability design formula

The new formula generally results in a much less costly design than the more conservative existing standard design approach in the USA and in a safer design than the existing standard design formula in Japan. Further details are given by *Madrigal and Valdés (1995).*

3.7 Wave overtopping

Comprehensive model testing and calibration by prototype behaviour have led to the evaluation of the effects of wave overtopping on pedestrians and vehicles on the breakwater, and thus to new admissible overtopping criteria. Moreover, a universal overtopping formula to predict the overtopping discharge as a function of the structure parameter and incident wave conditions has been developed.

The new formula applies for the full range of variation of the structure parameters, wave and depth conditions. This makes it possible to assess the performance of new constructional measures to reduce overtopping. Moreover, the formula generally results in a safer design because it yields larger overtopping rates than the existing standard methods. Further details are given by *Franco (1997)*. The effect of wave overtopping on the reduction of wave loading has been addressed by *Oumeraci and Kortenhaus (1997)*.

3.8 Three-dimensional effects on wave loading, overtopping and reflection

Since most of the design formulae are derived for normal wave incidence under 2D-conditions, a verification under 3D-wave conditions was necessary. Therefore extensive 3D-model tests for oblique and short-crested sea have provided correction factors which account for the effect of wave obliquity and short-crestedness of the waves on wave overtopping, wave reflection and wave loading.

Generally wave obliquity and short-crestedness cause less wave overtopping, reflection and loading than in the 2D-case. The reduction factors which have been determined will therefore allow to achieve less costly design than in the past. Further details are given by *Franco (1997)*.

4 Concluding Remarks and Perspectives

The integrated character of the research strategy which has been particularly purchased in PROVERBS as well as the multi-disciplinary research efforts to link together the hydrodynamic, geotechnical, structural and probabilistic design aspects represent a substantial departure from the existing, essentially empirical approach, and thus constitutes a very important step towards a largely rational discipline. In this respect, the results will not only enable to lay down the scientific basis for authoritative design guidelines for coastal structures, but also constitutes a scientific platform for continuous improvements, even after completion of the MAST Programme.

The process oriented research adopted in PROVERBS will also provide a physically sound departure and scientific basis for the development of innovative alternative constructions which can fulfil technical, economical and ecological criteria.

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The author would like to thank the Task leaders and all other partners of PROVERBS for their valuable contributions and for the new and enjoyable research atmosphere they created within PROVERBS. Thanks are also due to the European Communities for the support of these projects. The nice cooperation with the EC-representative C. FRAGAKIS is also acknowledged. The support of national research projects by the respective national research foundations and governments which largely contributed to the success of PROVERBS is also gratefully acknowledged. Without the national projects supported by the Deutsche Forschungsgemeinschaft (DFG) and the Bundesministerium für Forschung und Bildung (BMBF) the author would have been unable to efficiently contribute to PROVERBS.

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Annex A

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Topic IV

Harbours and Coastal Structures

Elements of Investigation

Chairman: Ming-Chung Lin

The Studies of the Instability of Hwa-Lian Harbour Caused by Typhoon Waves

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ABSTRACT

Hwa-Lian Harbour is located at the eastern coast of Taiwan, where is relatively exposed to the threat for waves from the Pacific Ocean. There is no adequate wave absorption facilities along the wharf and breakwater in the basin. In the summer season, waves generated by typhoon which locates at the eastern ocean of the Philippine, may cause down time for harbour operation and even damage the harbour facilities. In this studies, waves inside and outside of the harbour are measured at typhoon season, and wave characteristics together with movement of the berthing ships are analyzed.

The results indicate that inside the harbour short period wave energy reduced by sheltering of the breakwater, but long period wave energy increases several hundred times. Swells generated by typhoon which locates at the eastern ocean of the Philippine may cause down time for harbour operation.

1 INTRODUCTION

Hwa-Lian Harbour is an artificial harbour built on a steep and narrow east coast of Taiwan. More than 4000m long breakwater was constructed to protect harbour basin against incident waves from the Pacific Ocean. Figure 1 shows the location and layout of the harbour, which has 25 wharves after the fourth stage expansion. The length of the basin is about 4500m, and there is vertical wall inside the basin. In the summer season, waves generated by typhoon which locates at the east of the Philippine, may agitate harbour resonance, and cause down time for harbour operation.

Field investigation has been carried out since 1989, a waverider has been deployed at the outside of the harbour. In 21 June 1990, Typhoon Ofelia formed at the eastern ocean of the Philippine, and moved toward Taiwan in NNW direction. Finally made landfall with the eye of typhoon striking at 17km south of Hwa-Lian Harbour. One hour before landfall, the maximum wave height 20.5m, period 16.0 sec was recorded. The corresponding significant wave height and period are 13.9m and 14.1 sec respectively. Time series of wave height and period are plotted as shown in Figure 2. Four commercial vessels were berthed at the terminal, two of them escaped for survival when significant wave hight was greater than 4m; the other two ships broke the mooring lines and damaged harbour facilities while wave height increased rapidly.

In November 1992, there were 3 typhoons Elise, Hunt and Gay. The center of those typhoons were more than thousand kilometers away from Hwa-Lian Harbour at SE direction. But, the harbour basin was agitated by swells, and ships were forced to leave the unstable basin for survival.

In order to get a better understanding of the harbour hydraulic environment during the threat of typhoon, an intensive research program has been carried out by the Institute of Harbour and Marine Technology (IHMT) since march 1993. Field investigation, hydraulic model test and numerical computation are included in this research project. It is hopeful to obtain a feasible layout to eliminate harbour instability. This paper, present field investigation , wave characteristics, down time of harbour operation and their corresponding status of typhoon.

2 FIELD INVESTIGATION

Wave data recorded at the same time for outside and inside of the harbour are necessary to find out the mechanics of harbour instability and to evaluate the sheltering efficiency of breakwater. Figure 1 shows the location of wave stations. The permanent station, St.2 locates at outside of the harbour, a directional waverider and current meter have been installed since 1989.

At the harbour entrance,St.5 and terminal stations locate at water front of terminal #8 , #10 , #22 four pressure type wave gauges were set up when typhoon wave attacked Hwa-Lian Harbour.

At the offshore station, St.2 incident waves were recorded 20 min every two hours with sampling rate 1.28 Hz. At the harbour basin and entrance waves were recorded 34min every hour with 1 Hz sampling rate. There were six typhoons threatened Taiwan in 1994. Based on wave data recorded at the interval of typhoon and associated berthing conditions provided by Hwa-Lian Harbour Bureau, the location of typhoon that may cause down time for harbour operation are analyzed further detail in this paper.

3 WAVE CHARACTERISTICS

The paths of six typhoons which occurred in the summer of 1994 are plotted as shown in Figure 3. IHMT had succeeded to obtain wave data inside and outside of the harbour synchronously during Typhoon Tim, Fred and Gladys moved toward Hwa-Lian Harbour. Fig.4 shows time series of wave height variation for the offshore station St.2, the harbour entrance station St.5 and terminal stations #22 , #8 , #10. The sheltering coefficient is defined as the ratio of wave height between the terminal station or the harbour entrance station to the offshore station. The sheltering coefficient and corresponding incident wave direction recorded at St.2 are plotted as shown in the upper and middle diagrams of Figure 4. For a specific time, the location of typhoon center, the incident wave height and wave direction measured at St.2 and sheltering coefficients at the harbour entrance station St.5, terminal stations #22 , #8 , #10 are tabulated in Table 1.

TABLE 1 SUMMARY OF INCIDENT WAVES AND WAVE HIGH COEFFICIENTS

TYPHOON NAME	TIME (M/D/H)	CENTER LOCATION (KM)	WAVE DIRECTION (°)	INCIDENT WAVE Hs AT St.2 (m)	HELTERING COEFFICIENT ENTRANCE St.5	#22	#8	#10	(TYPHOON CENTER) NOTE
TIM	07/09/ 18:00	SE (600)	125	2.0	0.65	0.33	0.17	0.1	MOVED NW
	07/10/ 14:00	SE (200)	130	7.0	0.55	0.32	0.15	0.1	BEFORE LANDFALL
	07/10/ 16:00	SE (140)	130	10.2	0.49	0.27	0.15	0.1	BEFORE LANDFALL
FRED	08/19/ 00:00	ESE (1000)	115	2.6	0.52	0.30	-	0.1	MOVED WNW
	08/20/ 18:40	E (200)	95	4.9	-	0.19	-	0.1	MOVED NNE
	08/21/ 02:00	ENE (250)	75	2.9	0.45	0.38	-	0.11	MOVED NNE
GLADYS	08/31/ 08:00	ESE (700)	115	1.2	0.45	0.1	-	-	MOVED WNW
	09/01/ 00:00	ESE (300)	100	2.4	0.67	0.2	-	0.09	MOVED TOWARD H.L.
	09/01/ 10:00	E (60)	80	4.4	0.36	0.18	-	0.13	BEFORE LANDFALL
	09/01/ 14:00	WNW (100)	145	2.9	0.35	0.1	-	-	AFTER LANDFALL CHANGE WIND DIR.

Table 1 indicates that the sheltering coefficients of harbour entrance and terminal stations depend on the incident wave direction and sheltering environment. The incident wave direction have good relationship with the location of the typhoon center. For example, the center of Typhoon Tim was 600km in SE of Hwa-Lian, incident wave came from 125° , wave height coefficient reduced from 0.6 at the harbour entrance to 0.1 at the inner basin terminal #10.The center of Typhoon Fred located at 1000km in ESE direction and moved toward Hwa-Lian in WNW direction, wave height increased from 2.6m to 4.9m , incident wave direction decreased from 115° to 95°, the sheltering coefficient decreased accordingly. But, when the center of typhoon approached Hwa-Lian, the direction of incident wave and sheltering coefficients were affected local wind and changed rapidly.

Energy spectra for waves recorded at the permanent station St.2 and terminal stations #22 , #8 , and #10 are plotted as shown in Figure 5. After the sheltering of the breakwater, the short period wave energy measured at terminal stations were reduced tremendously. The degree of energy density reduction depends on the sheltering condition. At the outer basin the energy density ratio between terminal #22 and incident wave is about 1/10; at the inner basin the ratio between #8 or #10 and incident wave decreases further to 1/100. On the other hand, long period waves from 140sec to 160sec recorded in the harbour basin energy density were amplified several hundred folds. The detail time series water surface for various wave stations are plotted in Figure 6, which shows water surface recorded at harbour stations have rather long periodic motion in comparison with that of incoming wave recorded at St.2.

4 DOWN TIME ANALYSIS

According to the berthing records of Hwa-Lian Harbour indicate that in the summer season, down time for harbour operation due to the action of typhoon wave has occurred at least once every year since 1985. Based on the status of berthing conditions provided by Hwa-Lian Harbour Bureau from 1990 and the corresponding paths of the typhoon, as shown in Figure 7, issued by the Central Weather Bureau. Table 2 shows the summary of typhoon generated swells that made harbour instability and caused down time for harbour operation.

TABLE 2 SUMMARY OF DOWN TIME FOR BERTHING CONDITIONS AND CORRESPONDING TYPHOON STATUS

(1) DATE (YR/MON)	(2) TYPHOON NAME	(3) DOWN TIME FOR BERTHING CONDITION				(5) INITAIAL CONDITION FOR		
		TIME (D/HR)	WHARF NUM	SHIP TONNAGE (DWT)				
90/06	OFELIA	22/12:30 13:30	#19 #10	30,700 3,800	19.5 N 122.6 E (500)	21/20:00	17.4 N 124.3 E	780
90/08	BECKY	26/11:30 14:00	#19 #6	12,800 8,000	18.5 19.3 (570)	26/00:00	19.5 125.5	650
90/08	YANCY	17/16:20 17:30	#14 #17	4,000 8,400	20.8 127.0 (570)	17/00:00	19.8 128.0	820
90/09	DOT	06/20:00	#4	2,000	20.5 126.4 (600)	06/08:00	18.9 130.0	050
91/10	RUTH	26/**	#18 #15 #14	19,300 36,000 4,000	18.5 125.0 (700)	26/08:00	17.6 127.6	950
92/09	TED	21/** 22/**	#18 #19 #11	19,300 36,000 6,700	19.8 122.0 (450)	20/12:00	18.0 126.0	850
92/11	ELSIE	06/13:50 18:30 18:50 20:50	#17 #8 #20 #19	- - 35,000 27,000	24.0 134.0 (1300)	05/02:00	16.0 137.0	1800
92/11	HUNT	20/15:00 19:00	#19 #25	- -	20.5 139.5 (2000)	19/02:00	15.5 143.0	2400
92/11	GAY	26/04:00 04:00 10:00	#11 #10 #20	- 7,600 -	18.5 131.5 (1200)	24/08:00	13.8 139.5	2200
93/06	KORYN	27/**	#8 #4 #14 #10 #18 #21	2,800 11,900 6,000 8,100 7,600 5,600	EAST OF 120°	25/08:00	15.1 126.1	1050

Table 2 (continued)

93/08	TASHA	18/**	#19	31,200	19.0 123.0 (550)	17/14:00	17.8 124.3	820
93/08	YANCY	01/**	#19 #15 #20 #5 #10 #11	20,000 11,400 36,000 4,800 10,600 9,300	21.8 126.0 (500)	25/08:00	20.7 128.5	840
93/09	ABE	11/23:00	#19	18,600	21.0 122.3 (300)	11/14:00	20.5 122.8	420
94/07	TIM	10/02:30 07:00 08:20	#15 #23 #17	5,700 25,000 14,900	19.9 124.3 (500)	09/09:00	18.0 127.0	850
94/08	CAITLIN	03/16:20 17:20	#24 #10	35,800 7,600	22.5 122.0 (150)	03/06:00	20.6 124.0	470
94/08	DOUG	06/15:30 16:00 17:00	#23 #22 #24	12,400 13,700 35,800	18.5 126.0 (750)	06/02:00	17.2 128.1	950
94/08	FRED	19/17:00 17:30 17:40 18:20	#6 #19 #20 #24	2,500 12,900 9,700 35,800	22.2 126.6 (550)	19/02:00	20.8 129.0	840
94/08	GLADYS	31/18:10 19:40 20:20	#10 #25 #3	13,500 36,100 2,500	22.8 125.5 (450)	31/02:00	22.0 130.5	1000
94/10	SETH	08/07:20 08:10	# 6 #22	2,800 9,300	19.0 125.9 (700)	07/08:00	16.5 129.5	1150

NOTE: /** TIME UNKNOW

According to typhoon advance path, the leaving location can be determinded and listed in column 4 of Table 2. The leaving location is defined as the location of typhoon, while berthing ship commence to leave the harbour by agitation of swells. By assuming the propagation speed of swell is 50 km/hr, column 5 of Table 2 lists the estimation results of initial time and location, that typhoon generated swell might force ships to leave the harbour for safety. The analysis results are plotted as shown in Figure 8, which indicate swells generated at the initial location had arrived the harbour, while the center of typhoon moved from the initial location to the leaving location.

In the summer season, there are many typhoons occur in the Pacific Ocean. For those typhoons with low pressure center form at southeastern ocean of Hwa-Lian Harbour and travel northwestward, at the right rear quadrant will run with the cyclone. This will prolong the influence of cyclonic wind and build up wave height. Although short period wave energy is sheltered by the protection of breakwater, but the implicit long period small amplitude wave energy is enlarged hundred times by harbour geometry. Hence, typhoon occurs at the eastern ocean of the Philippine and moves northwesterly towards the east coast of Taiwan. At the offshore of Hwa-Lian, wave energy can be built up rapidly within one day. Under this condition, the generated swells might make harbour instability and cause down time for harbour operation.

5 DISCUSSION

The geometry of the outer basin of Haw-Lian Harbour is like a triangle. Although wave absorption perforated chamber was built along the wharf. But the depth is not enough to absorpt long period wave energy. In the summer typhoon season, swells generated at the eastern ocean of the Philippine. The harbour basin becomes unstable under the action of long period wave from southeast direction and associated agitation.

At the offshore permanent station a waverider has been employed to record wave data. Because the sensor of accelerator can not detect long period water surface motion. Hence, wave spectrum of the permanent station shows no extraordinary long period wave energy component. A pressure type wave gauge will be deployed in the further investigation program.

There are two reasons that berthing ship intend to leave harbour for safety. The first one is the harbour basin resonated by waves come from southeast direction. The terminal is unstable for mooring ship. Ship further stay in the harbour may damage facility and cause serious disaster. The second reason is due to psychological impact. Large scale typhoon moves toward Harbour directly, and wind speed increases rapidly. Even though wave climate in the harbour basin is not threatened by incident waves.

In order to find a feasible layout to eliminate long period wave energy, hydraulic model test and numerical computation are under execution.

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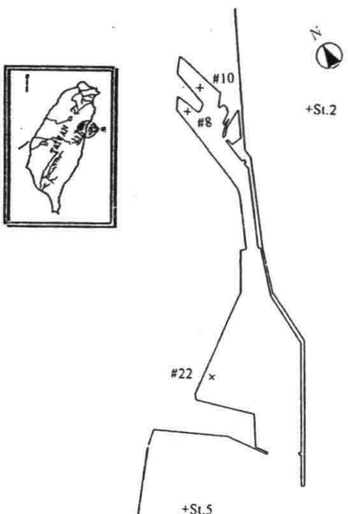

Figure1 Layout of Hwa-Lian Harbour and Location of Instruments

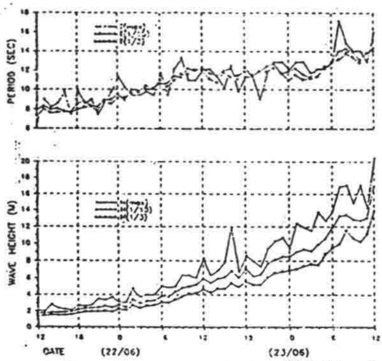

Figure2 Time Series of Wave Height and Period for Ofelia Typhoon

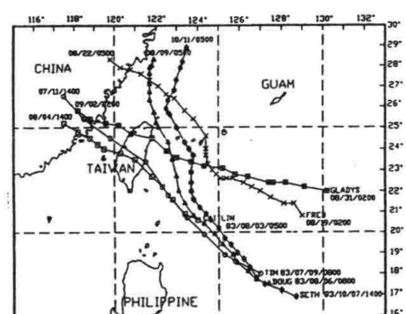

Figure3 Paths of 6 Typhoons Occurred in 1994

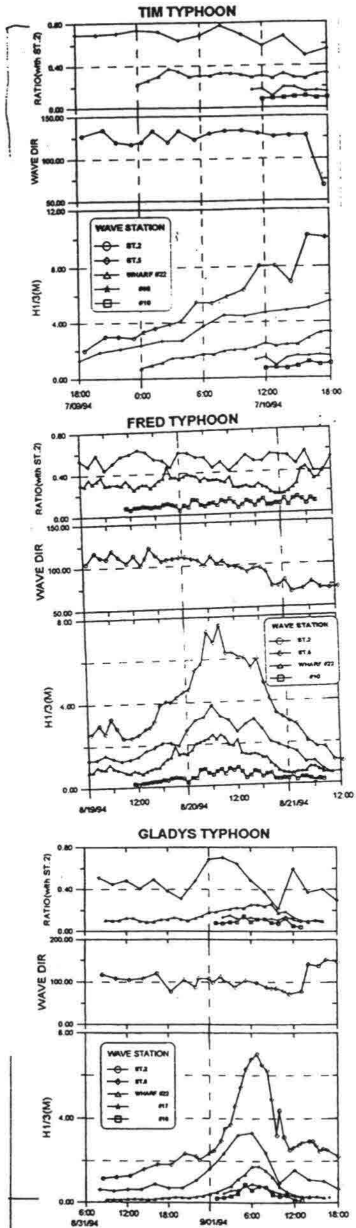

Figure4 Times Series of Wave Height Characteristics for Stations Insideand Outside of the Harbour

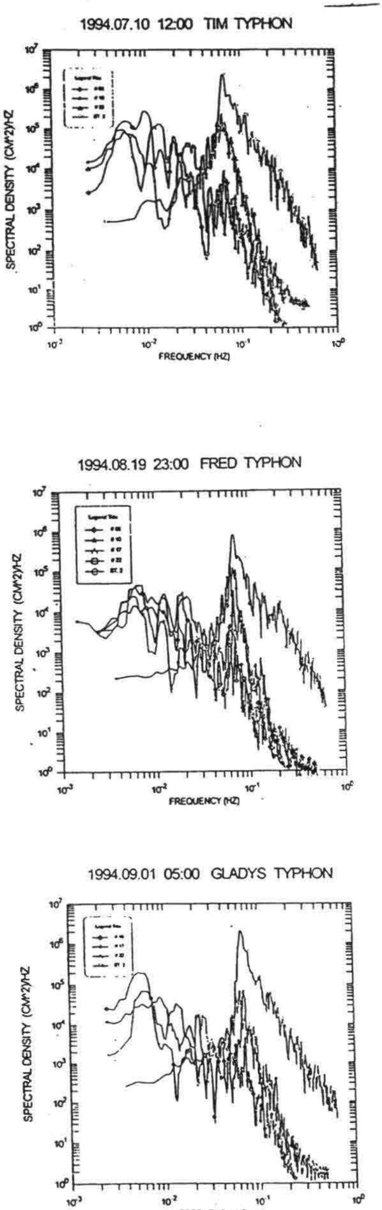

Figure5 Energy Spectra for Stations Inside and Outside of the Harbour

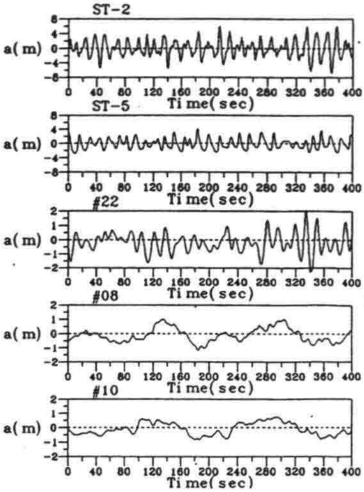

Figure6 Detail Time Serces of Water Surface Motion for Stations Inside and Outside of the Harbour

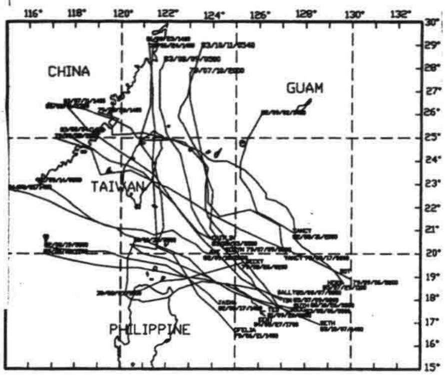

Figure7 The Paths of Typhoon that Have Caused Down Time for Harbour Operation Since 1990

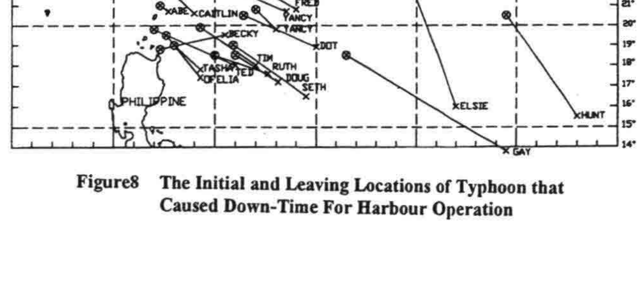

Figure8 The Initial and Leaving Locations of Typhoon that Caused Down-Time For Harbour Operation

Elements of an Integrated Harbour Design

The Functions of a Consulting Engineer

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Abstract

While carrying out the projecting, layout, technical design, construction and implementation and, if necessary, the operation of a harbour, the Consulting Engineer has to deal with an unusually large number of specialised fields.

He prepares the traffic forecast and compiles all the relevant information on the physical environment. He investigates the demands in the field of port operation and makes recommendations regarding institutional and organizational matters. He elaborates a long-term development plan and defines the measures to be executes within the first phase.

He prepares the environmental and social impact assessment and provides the economic and financial analysis. During the implementation phase he prepares the detailed designs, provides advisory services concerning tendering and awarding procedures and supervises the construction works.

If necessary, he runs the operation for a certain time after completion.

1 Introduction

My remarks refer primarily to planning for the construction of new harbours as well as to the extension and remodelling of existing harbours in developing countries, financed by bilateral or multilateral organizations, but in principle they also apply to other harbour planning.

If you ask different experts on harbour planning what they consider to be the most important subject of the investigations you will receive highly differing answers, depending on the specialist field of the person you ask.

One expert, for example, will reply: The most important subject involves oceanographic and hydrographic problems, such as protection against waves, currents and siltation, as well as determining the wave forces on structures.

Another expert will reply: The most important subject is the question of which queuing system is to be used to calculate the waiting times of the arriving ships.

A third expert considers that the most important question is how the maintenance and repair of the mechanical equipment is to be organized and financed.

You can continue asking this question, and you will soon find out that the Consulting Engineer, who has to carry out the overall plan, has to deal with an unusually large number of specialised fields, of which some of the particularly important are to be mentioned briefly here.

2 Traffic Forecast

The basis of harbour planning is the traffic forecast, in which the future type and volume of cargo throughput and the passenger traffic is assessed.

The loading and unloading capacities as well as the storage capacities of the harbour installations have to be planned in such a way that no bottlenecks occur, but also avoiding surplus capacities that are too large.

The development of the quantities of goods has to be determined for at least the following types of goods:

- conventional general cargo
- containers
- liquid bulk
- dry bulk.

Here a distinction has to be made between incoming and outgoing traffic as well as, if necessary, between origin and destination.

In this connection the establishment of hinterland development plans is necessary in most cases. These contain estimates of the economic potential of the hinterland in terms of population, income, industrial and agricultural production. If there are several possible means of transport, then the expected breakdown of the traffic flows into road, rail and inland waterways, i.e. the modal split, has to be determined.

It is useful to carry out the investigations for different prognoses (e.g. optimistic, medium, pessimistic), and to determine the influence of different assumptions on the results by means of sensitivity analysis.

3 Physical Environment

Examinations have to be carried out with regard to the availability of the land that is foreseen for the realisation of the planned harbour installations. The current owners must be determined, and the date of the termination of existing leases has to ascertained.

The current use of the land has to be taken into account during the planning, as well as the question of whether or not existing plants have to be demolished and whether resettlement is necessary. Port structures already in existence have to be assessed with regard to their structural state and to the possibility of using them commercially. This also applies both to traffic links with the hinterland and to the traffic networks in the hinterland. Further information has to be obtained on access from the sea, e.g. the water depths, nautical aspects and the system of navigational aids.

Topographical surveys of the site have to be made, and data has to be gathered about the climate and meteorology (e.g. temperature, humidity, winds, rainfall). The hydrographic and oceanographic conditions have to be obtained form the available sources, and they have to be complemented and reexamined by means of specific measurements on site; these include, for example, bathymetry, tides, water levels, waves, currents, sediment transport, littoral drift, salinity and the quality of the water.

The geotechnical data must be obtained for the technical design of the structures, roads and storage areas, as well as for land reclamation and dredging. These data concern the geological conditions, soil quality, sources of filling material for possible exploitation of quarries (stone blocks for breakwaters, aggregates for concrete production).

4 Port Operations

The task of the Consulting Engineer is to propose the cargo-handling operations and the despatch of passengers, taking into account technological changes in shipping and cargo-handling methods (e.g. containerisation, roll-on roll-off ships) and the development of ship sizes. He must give the port performance indicators, such as the waiting time/service time ratio, berth throughput and occupancy, gang and ship output for typical commodities and types of operation, the impact of indirect and direct handling systems and the need for handling equipment. He must also put forward proposals for the pre-planning, monitoring and performance review of stevedoring, quay transfer, storage and receipt/delivery operations for container, roll-on roll-off ships, conventional general and bulk cargo, including the allocation of the berths, storage areas, port equipment and labour. The typical composition of a gang, their productivity, the working hours and the number of shifts have to be defined.

The loading and unloading capacities must be ascertained, information has to be provided on when the individual capacities will be exhausted, and which measures will then have to be taken. This information can refer to the capacity of the quay length (reaching the acceptable maximum berth occupancy), the capacity of the handling equipment and the capacity of the temporary storage areas or volumes.

5 Port Institutions, Organization

It has to be determined under which ministry, or ministries, the port administration works. In those cases in which the port services are largely privatised, the administrations tasks can be restricted - in extreme cases to the tasks that must, from the constitutional point of view, be performed solely by the government. The tasks of the port administration must be clearly defined and clearly distinguished from the tasks of the port operating company.

The structures of the port administration and of the port operating company have to be defined, including the demands for management and human resources. Proposals for the participation of the private sector, also within the framework of the ancillary services such as pilotage, towage, mooring, dredging, port security, bunkering, canteens and rest-rooms, communications, garbage collection and waste disposal have to be worked out. Special attention has to be paid to engineering as well as to the maintenance and repair services.

Within the framework of the port organisation it has to be guaranteed that the policies for port operations are continually updated, including tariff-setting, labour supply and contracts for cargo handling. Cooperation and communication with shipping and forwarding agents, trucking companies, shippers and receivers has to be assured with the aim of guaranteeing optimized ship and gang output.

6 Development Plan, Project

The Consulting Engineer has to recommend a phased port development programme (master plan) meeting short, medium-term and long-term requirements. The master plan includes the physical development plan, which comprises the layout of the port, giving the existing facilities and the proposed land use and installations, and showing their principal dimensions, the sites used and the relevant development stages. The proposed traffic flow and stacking arrangements are to be indicated. The physical plan is to be supplemented by estimates of the future potential throughput and storage capacities of the port.

It is usual that the first phase of the master plan forms the project to be examined in greater detail. The planning for this is carried out on the level of a feasibility study, i.e. the individual plants and items of equipment are not yet planned in detail (this is kept for the implementation phase, see below), but enough basic technical planning is done to enable a preliminary cost estimate to be made.

The plans for the project contain the representation of the technical facilities, such as breakwaters, quay walls including mooring and fender systems, transit sheds, passenger sheds, offices, workshops, traffic and storage areas, installations such as water and power supply systems, lighting and firefighting systems, waste reception and treatment facilities etc. The dredging requirements (capital dredging, maintenance dredging) as well as the necessary water depths have to be determined.

The definition of the project also includes the necessary handling and transport equipment.

7 Environmental and Social Impact Assessment

The following have to be examined within the framework of the environmental impact assessment:

- effects of the location
- effects of the preconstruction phase
- effects of the construction
- effects of the expected operation

It has to be guaranteed that national and international legislation, regulations and standards are taken into account in the planning, construction and operation of the facilities.

Special attention has to be paid to the effect of dredging works, both during the construction phase (capital dredging) and also during the operation phase (maintenance dredging). Here the effects directly at the location of the dredging and the question of the site for dumping and disposal have to be taken into account. As a general point, the impact on the ecology, fishery and land use, and the question of water pollution and/or erosion have to be investigated.

For the operation phase it is important that during the planning of the project sufficient attention is paid to the questions of dust arising during the loading and unloading of dry bulk, to garbage collection and waste reception and treatment facilities, and that the handling, storage and transport of dangerous goods can be carried out in accordance with the regulations.

Social impact occurs when the population in the area in question has to be resettled, and then the question of compensation arises. In this connection the effects on historical, cultural or religious monuments and sites around the port can also play a role.

8 Economic and Financial Analysis

The internationally operating financing organizations have each drawn up guidelines for carrying out the economic and financial analysis. These guidelines are similar to each other all over the world, and the Consulting Engineer has to abide by them in his investigations. The basis is formed by the determination of the costs and by the definition of the benefits.

The cost estimates contain:

- investment costs, broken down into foreign and local costs
- running costs
- costs for engineering
- costs for contingencies
- costs for price escalation
- possibly: interest during construction

The time schedule of the expenditure has to be worked out.

The benefits can, for example, arise from:

- cost saving when compared with more expensive handling procedures, e.g. handling at a quay versus handling by lighterage
- savings in transport costs in the case of goods that would have to be transported at greater expense by other means if the project were not realised (diverted transport).

An important influence is exerted, of course, by the loan conditions, i.e. the fixing of the interest rate, the repayment period and the grace period.

9 Implementation

When the investigations briefly mentioned in the previous sections have been gathered together in the FEASIBILITY STUDY and have led to a positive result, and if the funding has been guaranteed, the next step is then the implementation stage.

In this phase the Consulting Engineer has the following tasks:

- carrying out additional site investigations at the project site in order to determine the conditions for detailed design purposes
- preparing detailed designs and specifications for the civil works of the project
- assisting the master of the works with contractor prequalification
- preparing tender documents for the individual construction and equipment supply contracts
- providing assistance to the master of the works during the tendering process, including the answering of tenderers´ questions, attendance at pre-tender meetings and evaluation of the offers
- providing assistance to the master of the works in contract negotiations and preparation

- supervising the construction works on site, including acting as „The Engineer“ in accordance with the FIDIC Regulations (Fédération Internationale des Ingenieurs-conseils, Lausanne)
- carrying out inspections at the premises of equipment manufacturers, and supervising and inspecting the installation, testing and commissioning of equipment
- possibly running the facilities for a certain time after completion, and giving further technical assistance, if necessary.

10 Closing Remarks

The many different tasks of the Consulting Engineer can only be carried out by a team which, like a orchestra, is comprised of a large number of experts, each of whom has mastered his field. However, each one of them ought to have an open ear for the other members of the team. The conductor, i.e. the project manager, is responsible for the efficient cooperation of everyone, and he ought to have an overview of the entire project and minimize dissonance within the team.

Extension of the Navigation Channel to the Habour of Rostock.
Investigations for the Design of Breakwaters and Entrance Channel

Gerd Flügge - Holger Rahlf - Norbert Winkel
Federal Waterways Engineering and Research Institute, Coastal Department, Hamburg

The future development of shipping traffic (bulk carriers and ferries) to Rostock habour will necessitate an extension of the entrance (deepening and widening) of the navigation channel. Therefore a new breakwater system is required.

Special conditions in this area must be considered:

- recreation, tourism, watersports and landscape criteria
- nautical conditions for large-scale shipping
- environmental impact.

By order of the local authority (WSA Stralsund) hydraulic model investigations to determine the functional design and the ship-induced loading (waves and currents) were carried out at the BAW (Federal Waterways Engineering and Research Institute) in the scale 1 : 40.

Aims of the Investigation

The purpose of this investigation was to develop a breakwater system which would allow ferries and vessels with up to 250 m length, 40 m beam, and 13 m draft to reach the harbour of Rostock.

Figure 1 depicts the coastal area showing the situation in 1990. The navigation channel is only 80 m wide and approximately 13 m deep at the narrowest point. Extension of the channel to a width of at least 120 m and a depth of 14.5 m is planned. Both shorelines of the channel are developed so that any expansion must occur within the present system, i.e., the center breakwater must be removed. Thus an entirely new breakwater system must be developed.

The new breakwater system must fulfill the following requirements:

- protection from rough seas coming from various directions, especially for the harbour area of Warnemünde;
- favorable nautical conditions, e.g., minimization of the banking effect;
- improved traffic regulation;
- acceptable ship induced loads (resulting from primary and secondary waves as well as countercurrents) in the Warnemünde harbour area;
- no detrimental change to the present coastal longshore current;
- no appreciable change in the landscape such as obstruction of the view to the Baltic, no detrimental effects on tourism;
- economical construction and maintenance

Approval of the expansion plans dictates that environmental protection be guaranteed with respect to possible changes to water exchange processes, current and salinity conditions, and the effects of storm surges for the Warnow estuary. This is especially important with respect to use of the Warnemünde harbour area.

Hydraulic model, breakwater systems and wave conditions

The geometry of the system and thus the resonance conditions resulting from sea and ship wave loading are very complex so that a hydraulic model with a scale of 1 : 40 was deployed (Figure 2).

Preliminary investigations were done using a numerical refraction model. Figure 3 shows the wave rays for peak periods of 5, 7, and 9 seconds for various directions of approach. The results show the effect of the deep navigation channel on the refraction of the waves. Waves coming from north 30° west are bundled in the area at the head of the West Breakwater. This phenomenon is observed in the model experiments as well as in nature. For waves coming from the west, the refraction at the deep channel causes a superposition of waves with a concentration of energy in the area of the western head of the breakwater. A part of the incoming waves can be effectively blocked by placing a transverse branch near the head of the West Breakwater (System 3).

Various systems for the breakwater configuration were chosen based on theoretical considerations and a numerical refraction model and are shown in Figure 4.

The existing West Breakwater is a tourist attraction. For this and for economic reasons, it will remain a part of the new breakwater system. There is a basic conflict in achieving the goals of providing the largest possible opening between the breakwater heads to accomodate shipping, but in turn providing a small opening to minimize wave intrusion. As a result of these criteria, the East Breakwater head was sloped to the navigation channel side. The form of the East Breakwater allows for an adequate distance between breakwater and channel in the inner system so that wave energy can be dispersed and an area is provided for recreational boating avoiding main channel traffic. An additional protection from sea waves for the Warnemünde harbour area is provided by the inner breakwater branch (System 1a, b and c). System 3 has a transverse structure at the head of the West Breakwater which effectively reduces the input of sea wave energy.

A design sea state was chosen on the basis of a wave climate study and maximum wave height analysis provided by the Geesthacht Research Center (GKSS). The methods used and the results achieved by the GKSS have been presented by GÜNTHER in this seminar. The breakwater system was functionally optimized using direction dependent sea wave events ("rough sea") with a probability of occurrence of 10 h/a. For designing the breakwater, an extreme event in the form of a severe storm surge with a high water level of NN + 1.85 m and a sea state with a probability of occurrence of 1 h/50 was chosen. A sea state having PIERSON/MOSKOWITZ-Spectrum (KITAOGORODSKIJ-Factor) corrected for shallow water was used in the model experiments. Waves were measured using sensors from the Danish Hydraulic Institute in conjunction with a computer supported data acquisition system (Disylab). Data were evaluated using FFT analysis and the ZERO-DOWN-CROSSING method in the time domain.

	Chance of Occurrence	Direction of Incident Wave				
		N60W	N30W	North	N30E	N60E
Rough Sea Conditions	10h/a Water Level: D = NN ± 0m	H_s = 3.1m T_p = 8.4s	H_s = 1.9m T_p = 6.0s	H_s = 1.8m T_p = 5.9s	H_s = 2.4m T_p = 7.0s	H_s = 2.0m T_p = 6.2s
Extreme Surge and Wave Conditions	1h/50a Water Level: D = NN ± 1,85m	H_s = 3.7m T_p = 9.4s	H_s = 3.7m T_p = 9.4s	H_s = 3.7m T_p = 9.4s	H_s = 3.7m T_p = 9.4s	H_s = 2.8m T_p = 7.8s

Tabelle 1: Spectral Characteristics to Classify Sea State

Results of the model experiments

The change of the wave state caused by refraction, shoaling and diffraction is shown in Figure 6.

When comparing the various systems, one must especially consider that the harbour area of Warnemünde is used by small ships and only small wave amplitudes are permissible. A definitely poorer wave height situation compared to that of 1990 can not be tolerated.

The construction of a new East Breakwater in connection with existing West Breakwater system (System 2) produced a definite wave amplitude reduction in the inner section compared with the wave amplitudes at the harbour entrance. However, the situation is definitely worse in the Warnemünde area for the measuring points 7 to 10. By placing an inner branch at the West Breakwater (System 1a), the waves at Warnemünde could be definitely reduced compared with present conditions. With respect to wave heights, System 1a provides the optimal solution. However, this case was rejected for nautical reasons. All the small ships would be forced because of this breakwater branch to enter the harbour using the main channel of the large ships. The large ships would also be hindered by this crossection reduction (unsymmetric banking effect). A shortening of the inner tranverse branch(System 1b) causes a reduction in the wave protection at Warnemünde.

After discussions with the pilots it was decided for nautical reasons to deploy only a transverse branch near the head of the West Breakwater as a counterbalance to the head of the East Breakwater (System 3). Through the use of this transverse branch, a definite reduction, compared to System 2, of the wave amplitudes at Warnemünde was achieved, especially for waves coming from north 30° west. Refraction of these waves at the deep channel cause a concentration of wave rays in the area west of the channel. The effectiveness of this breakwater branch could be substantially increased if it were stretched to the west edge of the channel. Again, this is not acceptable since all of the small ships would be forced to enter the harbour via the main shipping channel. System 3, with the short transverse branch, provides for a more favorable traffic situation. Only major shipping uses the main channel and the minor and recreational shipping use the area between the breakwater head and the western channel boundary for entering and leaving the harbour.

Figure 11 gives an overall picture of the breakwater systems with respect to the achievable wave damping in the inner Warnemünde Harbour (Gange A2). The relative wave heights are plotted against the initial wave heights. This figure clearly shows that the protection from a severe sea state significantly depends upon the sea state parameters and the water levels.

System 3 was then chosen for nautical reasons even though there is an increase of wave activity in some areas as a result of the wide entrance. Thus System 3 is a compromise between nautical requirements and the achievable reduction of wave amplitudes.

Breakwater design (profiles and material properties)

The highest water level ever recorded at Warnemünde was 2.43 m above NN. Water levels between 1 and 1.5 m above NN are expected with a probability of occurrence of one to ten years. The deciding factor for determining the height of the beakwater was, how well the breakwater fits into the landscape. The height was set to NN + 2 m in order to guarantee an unobstructed view of the Baltic Sea from the West Breakwater. This low breakwater height is geotechnically advantageous since an extensive foundation is not needed and the expected settling of 0.3 to 0.5 m can be compensated by increasing the initial construction height respectively. An overtopping of the waves for certain water levels and wave heights is deliberately allowed for. Since long crested waves will never arrive orthogonal to the East Breakwater, high energy resonances are not expected within the breakwater system. The profile of breakwater is constructed so that damage from breaking waves does not occur. Thus the height and profile of the wave breakwater was determined not only on the basis of scientific analysis but also on the basis of landscape, foundation, and economic criteria.

The inner and outer slopes of the planned East Breakwater are unusally flat with values of 1 : 3 and in some sections 1 : 4. This minimizes reflection effects to such an extent that special anti-scour measures at the base of the breakwater are unnecessary. The breakwater armor possesses a long time stability especially for obliquely approaching waves. Reflections at the outer slopes which could cause erosion at the adjacent beaches are not expected. The flat inner slopes guarantee minimal reflection of sea waves and especially of ship waves. Energy from breaking waves is dissipated in the surf zone - practically no inshore waves are generated. The flat sloped construction of the breakwater is less sensitiv to geotechnical settling.

Special attention was paid to the construction details of the breakwater head. The breakwater head experiences the superimposed effects of refraction, shoaling, diffraction and breaking from waves of various directions. The breakwater head is sloped 1 : 2 towards the channel. This area must have a very secure cover layer since any loose blocks landing on the channel bed would threaten shipping safety or, e.g., cause environmental damage if an oil spill occurred. Since the breakwater head is of cardinal importance, stability investigations were performed at the wave tank of the Danish Hydraulic Institute. These investigations showed that cover stability is guaranteed even for extreme sea states if the cover stones weigh 6o kN with a density of 30 kN/m^3. For the landward part of the East Breakwater, the stone weights were reduced shoreward from 40 to 25 kN corresponding to the reduced sea wave loading. Confirmation of these results were aided through the scientific support Prof. Kohlhase from the Institut für Wasserbau in Rostock and were based on the relevant literature. However, it turned out that the classic approach of HUDSON and the newer approach of VAN DE MEER did not provide an adequate theoretical basis for designing the breakwaters due to the complicated system geometry and wave directions. The design of the East Breakwater was carried out by using engineering type empirical estimates and scientific knowledge.

A special geometry was chosen for the West Breakwater. As was shown by the refraction diagram the deep channel effect, a substantial part of the sea wave energy is concentrated in this area. A wave energy absorbing - not reflecting - geometry was purposely chosen in order to avoid a worsening of the nautical conditions for the small shipping resulting from increased wave reflections. The area between the existing West Breakwater and the West Breakwater Finger is formed as a flat curved slope with a heavy cover layer. The approaching sea is dissipated in the surf zone of this area with a minimum of wave reflection occurring. A higher degree of wave damping could be achieved if the West Breakwater branch could be extended closer to the main channel. This, however, was not possible due to nautical reasons.

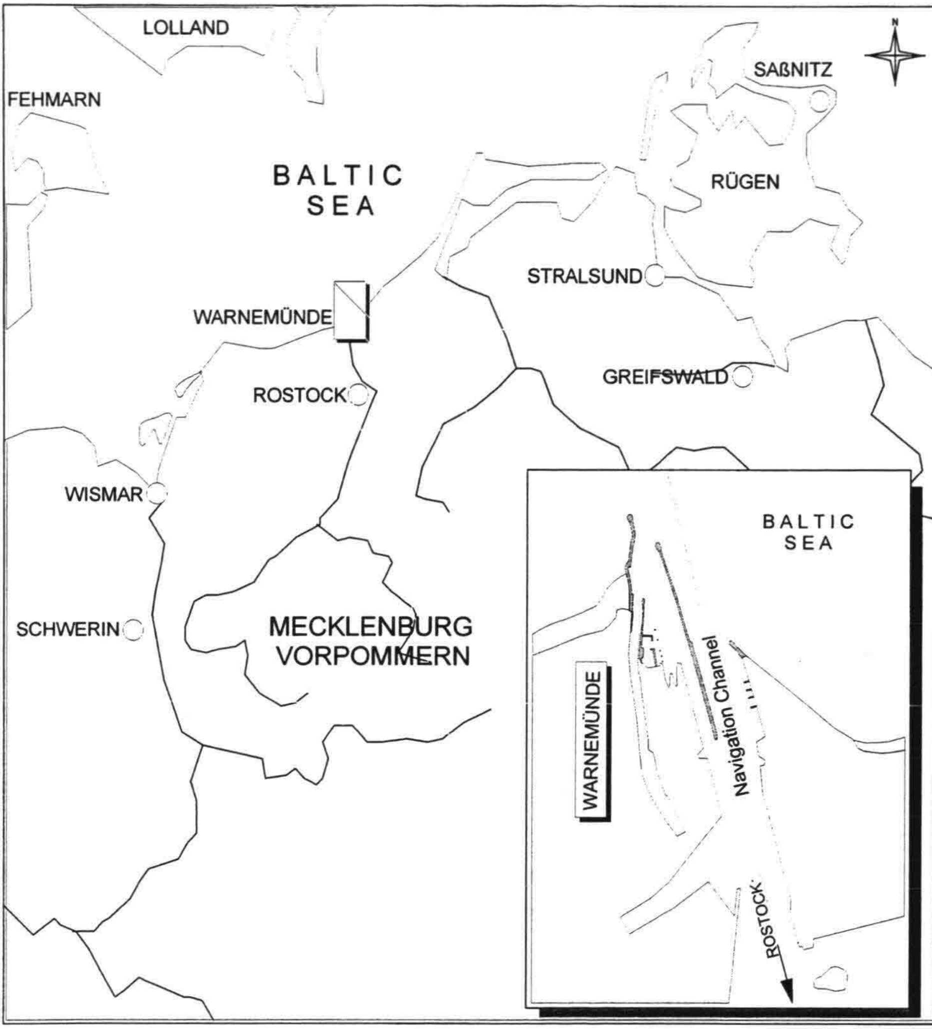


Fig. 1: Coastal area, situation in 1990

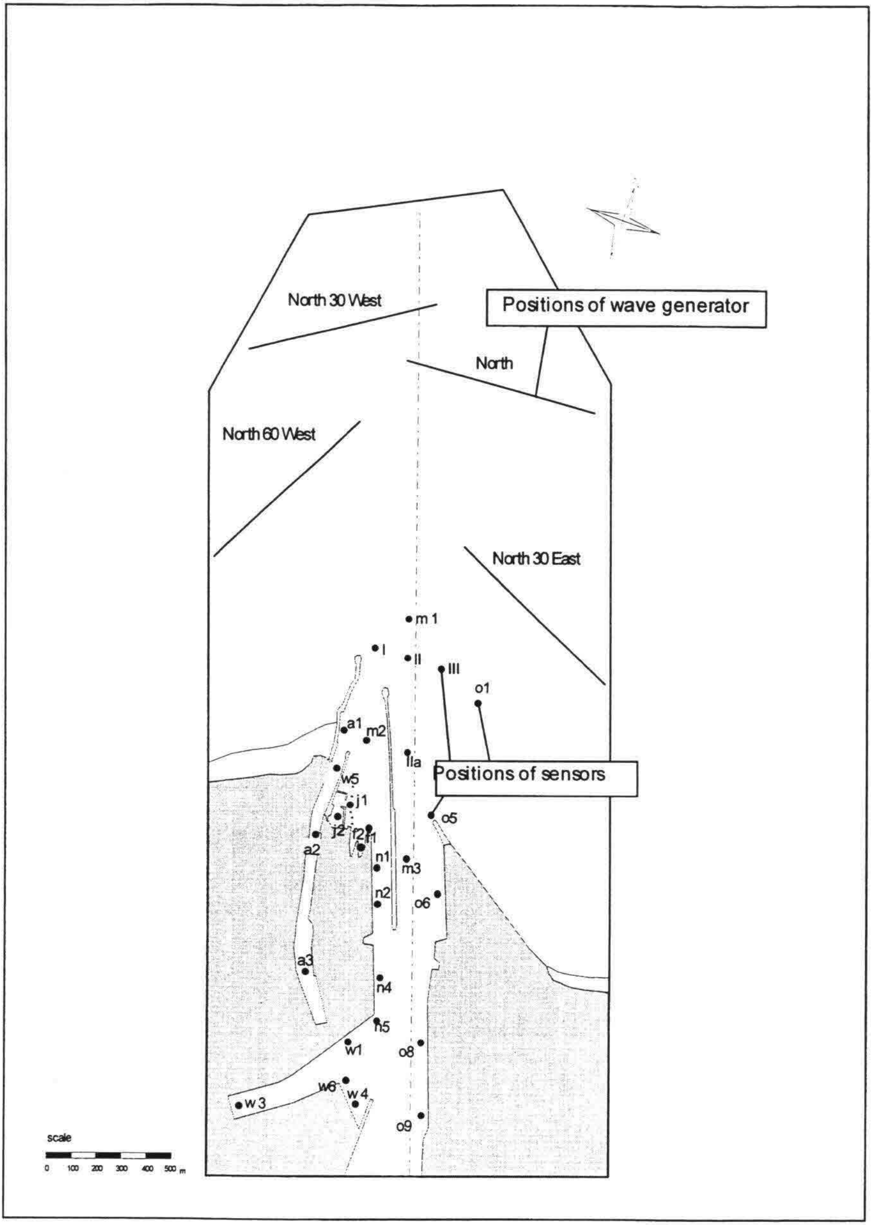


Fig. 2: Hydraulic model 1:40

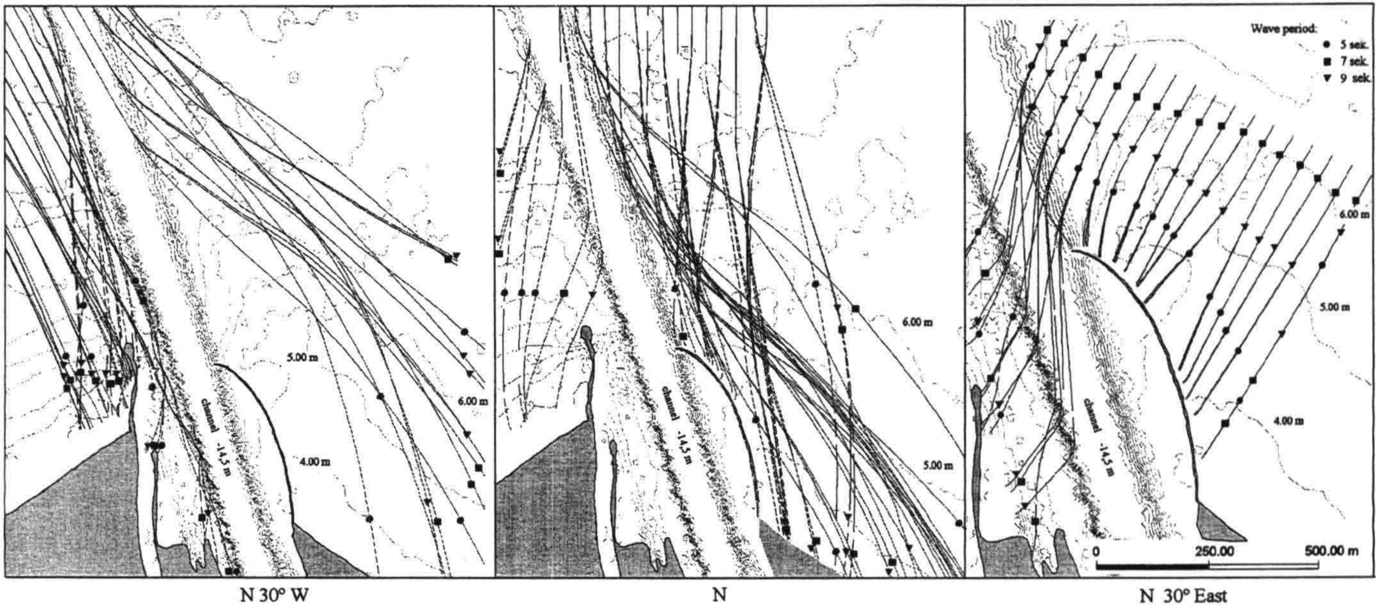


Fig. 3: Computed wave rays for three directions of the incident waves coming from N30° West, North and N30° East.
The symbol •denotes the rays of waves with a period of seconds, ■ 7 sec and ◢ 9 sec.

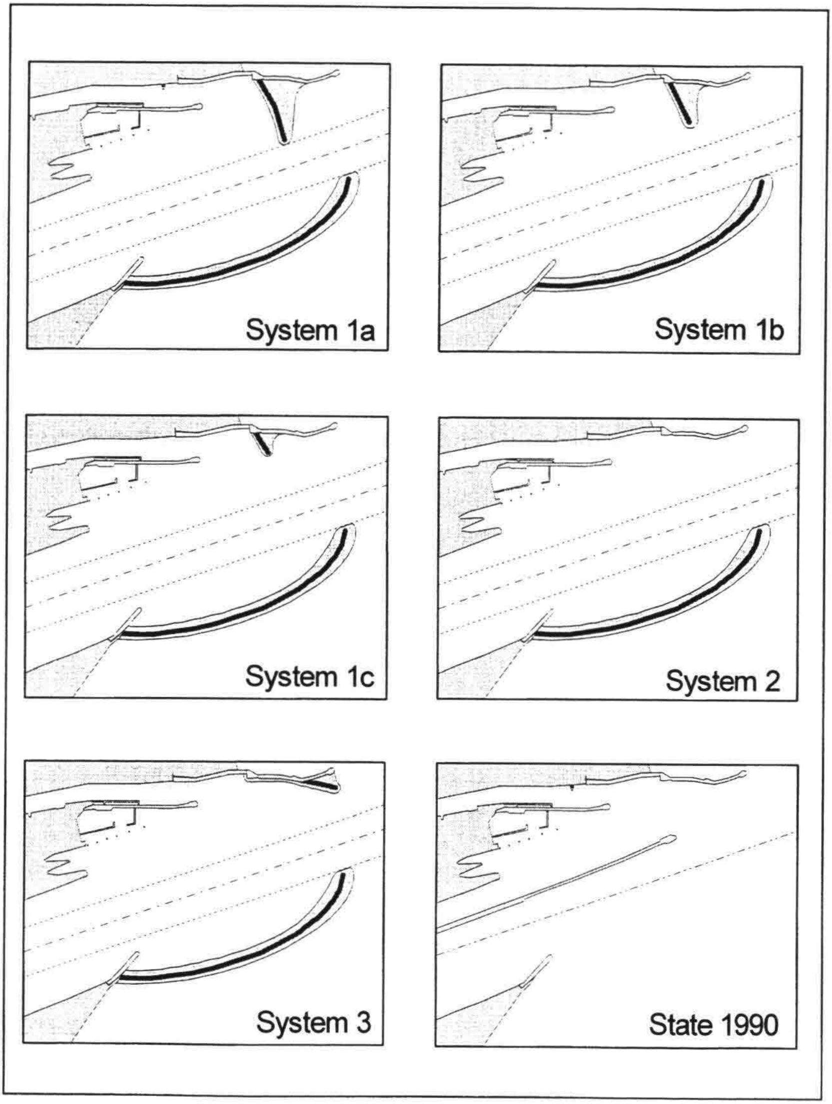


Fig. 4: Investigated breakwater systems

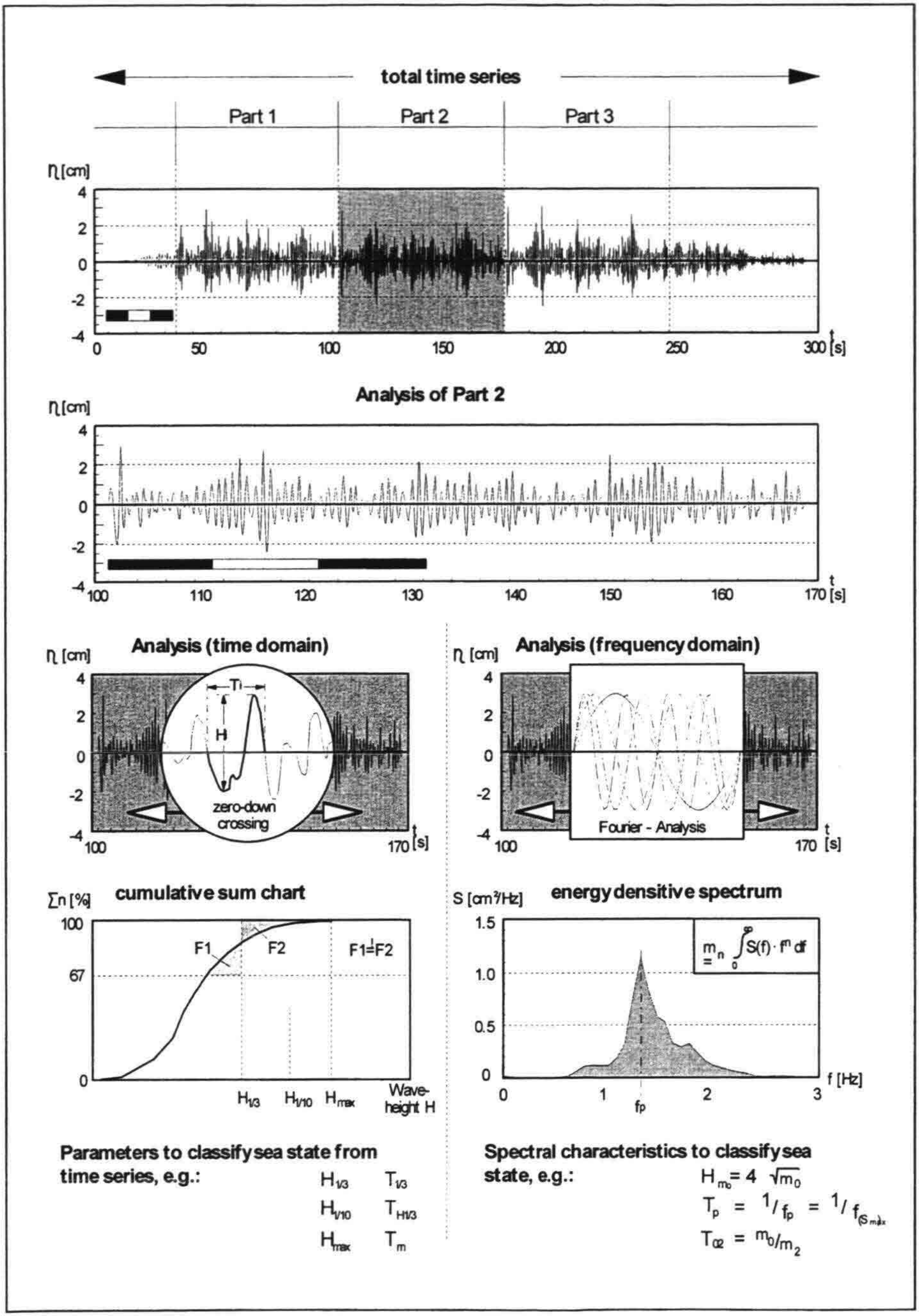


Fig. 5: Principle of data analysis

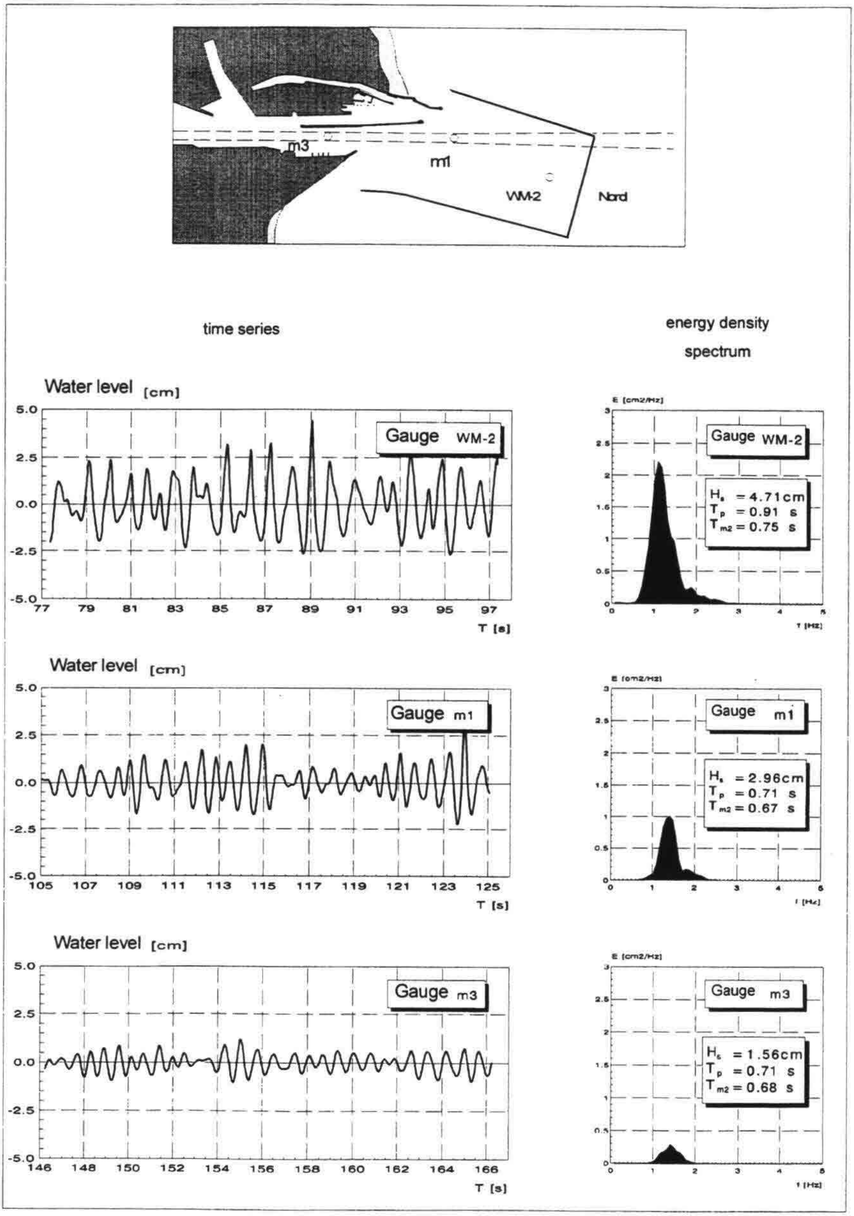
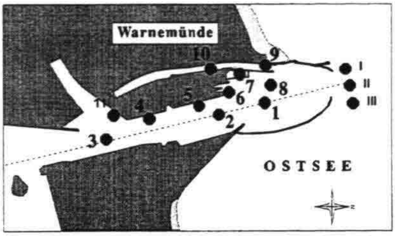


Fig. 6: Wavetransformation due to shoaling, refraction and diffraction



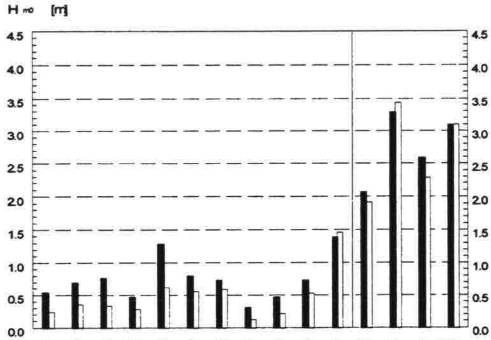
Rough Sea Conditions

Wave parameters
(generated)

Direction

N 60° W

H_{m0} = 3,1 m
T_p = 8,4 s
P = 10h/a
W = NN

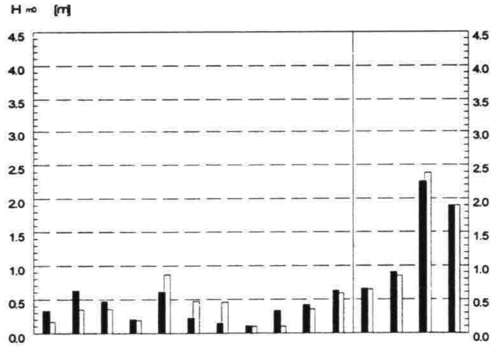


	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.6	0.7	0.8	0.5	1.3	0.8	0.7	0.3	0.5	0.7	1.4	2.1	3.3	2.6	3.1
State 1990 □	0.2	0.4	0.3	0.3	0.6	0.6	0.6	0.1	0.2	0.5	1.5	1.9	3.4	2.3	3.1

Direction

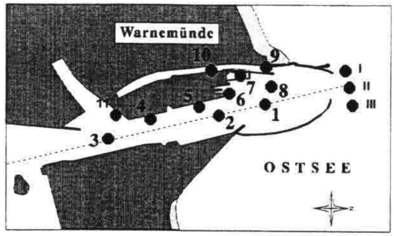
N 30° W

H_{m0} = 1,9 m
T_p = 6,0 s
P = 10h/a
W = NN



	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.3	0.6	0.5	0.2	0.6	0.2	0.2	0.1	0.3	0.4	0.6	0.7	0.9	2.3	1.9
State 1990 □	0.2	0.4	0.4	0.2	0.9	0.5	0.5	0.1	0.1	0.4	0.6	0.7	0.9	2.4	1.9

Fig. 7: Comparison of measured wave heights (State 1990 - System 3)



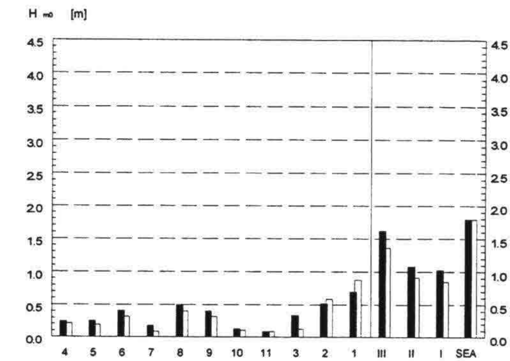
Rough Sea Conditions

Wave parameters
(generated)

Direction

North

H_{m0} = 1,8 m
T_p = 5,9 s
P = 10h/a
W = NN

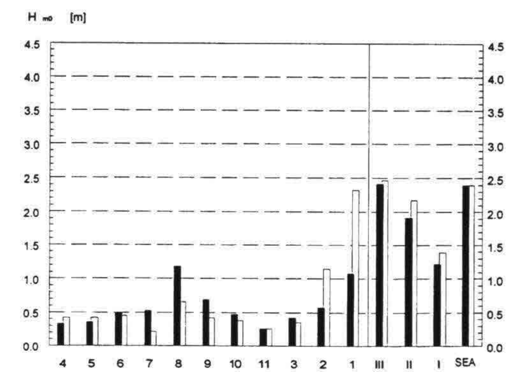


	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.2	0.3	0.4	0.2	0.5	0.4	0.1	0.1	0.3	0.5	0.7	1.6	1.1	1.0	1.8
State 1990 □	0.2	0.2	0.3	0.1	0.4	0.3	0.1	0.1	0.1	0.6	0.9	1.4	0.9	0.9	1.8

Direction

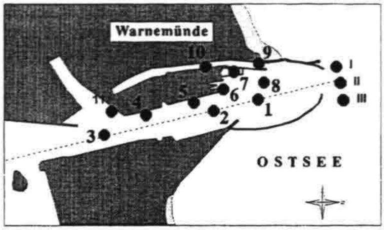
N 30° E

H_{m0} = 2,4 m
T_p = 7,0 s
P = 10h/a
W = NN



	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.3	0.4	0.5	0.5	1.2	0.7	0.5	0.3	0.4	0.6	1.1	2.4	1.9	1.2	2.4
State 1990 □	0.4	0.4	0.5	0.2	0.7	0.4	0.4	0.3	0.4	1.2	2.3	2.5	2.2	1.4	2.4

Fig. 8: Comparison of measured wave heights (State 1990 - System 3)



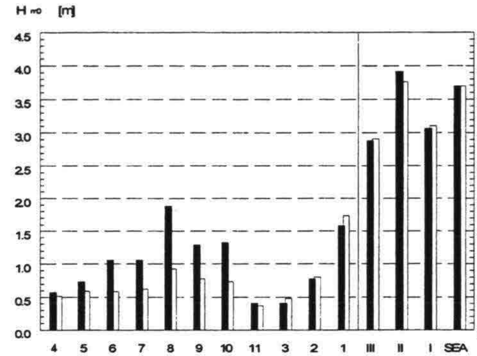
Extreme Surge and Wave Conditions

Wave parameters (generated)

Direction

N 60° W

H_{m0} = 3,7 m
T_p = 9,4 s
P = 1h/50a
W = NN + 1,85

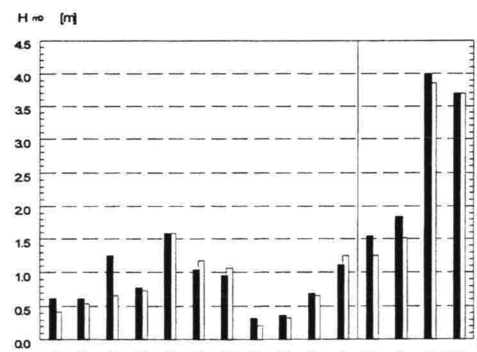


	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.6	0.7	1.1	1.1	1.9	1.3	1.3	0.4	0.4	0.8	1.6	2.9	3.9	3.1	3.7
State 1990 □	0.5	0.6	0.6	0.6	0.9	0.8	0.7	0.4	0.5	0.8	1.7	2.9	3.8	3.1	3.7

Direction

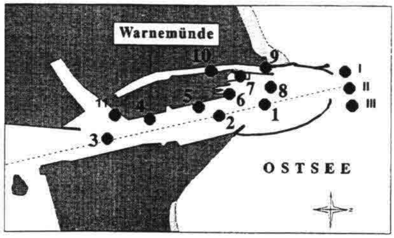
N 30° W

H_{m0} = 3,7 m
T_p = 9,4 s
P = 1h/50a
W = NN + 1,85



	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.6	0.6	1.3	0.8	1.6	1.0	1.0	0.3	0.4	0.7	1.1	1.6	1.9	4.0	3.7
State 1990 □	0.4	0.5	0.7	0.7	1.6	1.2	1.1	0.2	0.3	0.7	1.3	1.3	1.5	3.9	3.7

Fig. 9:Comparison of measured wave heights (State 1990 - System 3)



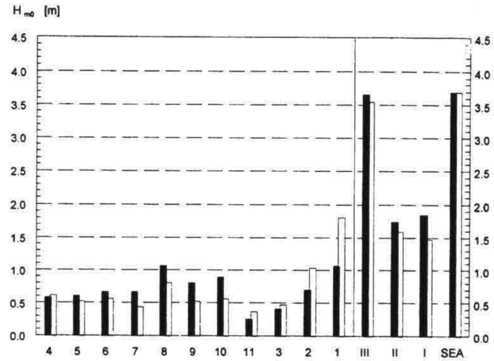
Extreme Surge and Wave Conditions

Wave parameters (generated)

Direction

North

H_{m0} = 3,7 m
T_p = 9,4 s
P = 1h/50a
W = NN + 1,85

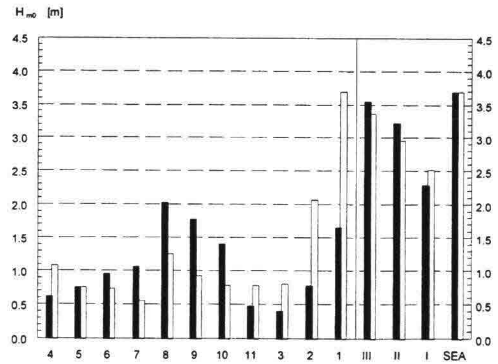


	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.6	0.6	0.7	0.7	1.1	0.8	0.9	0.3	0.4	0.7	1.1	3.7	1.7	1.9	3.7
State 1990 □	0.6	0.5	0.6	0.4	0.8	0.5	0.6	0.4	0.5	1.0	1.8	3.6	1.6	1.5	3.7

Direction

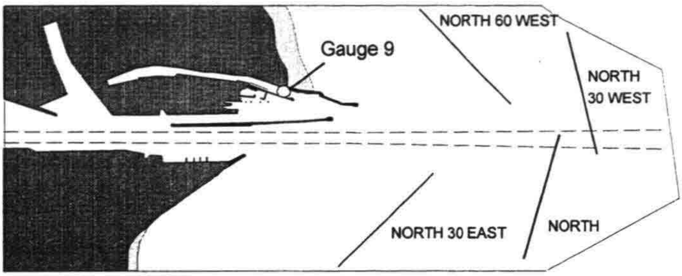
N 30° E

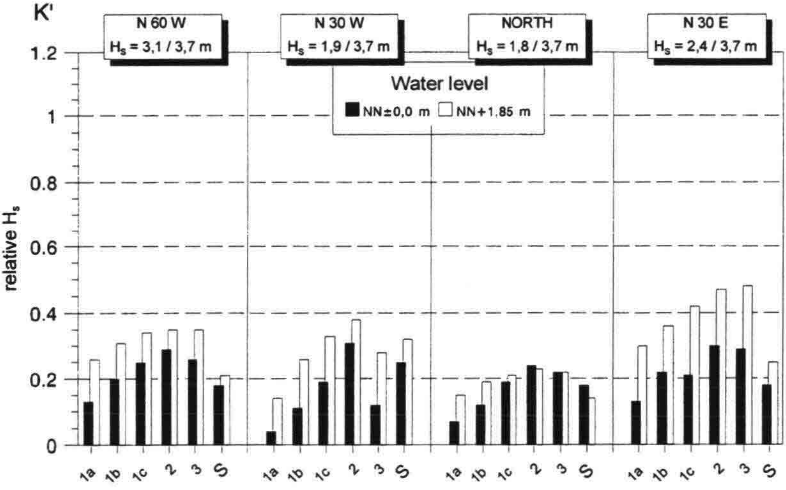
H_{m0} = 3,7 m
T_p = 9,4 s
P = 1h/50a
W = NN + 1,85



	4	5	6	7	8	9	10	11	3	2	1	III	II	I	SEA
System 3 ■	0.6	0.8	1.0	1.1	2.0	1.8	1.4	0.5	0.4	0.8	1.7	3.6	3.2	2.3	3.7
State 1990 □	1.1	0.8	0.7	0.6	1.3	0.9	0.8	0.8	0.8	2.1	3.7	3.4	3.0	2.5	3.7

Fig. 10: Comparison of measured wave heights (State 1990 - System 3)





NN±0,0 m	0.13	0.2	0.25	0.29	0.26	0.18	0.04	0.11	0.19	0.31	0.12	0.25	0.07	0.12	0.19	0.24	0.22	0.18	0.13	0.22	0.21	0.3	0.29	0.18
NN+1,85 m	0.26	0.31	0.34	0.35	0.35	0.21	0.14	0.26	0.33	0.38	0.28	0.32	0.15	0.19	0.21	0.23	0.22	0.14	0.3	0.36	0.42	0.47	0.48	0.25

Fig. 11: Comparison of relative wave heights K'

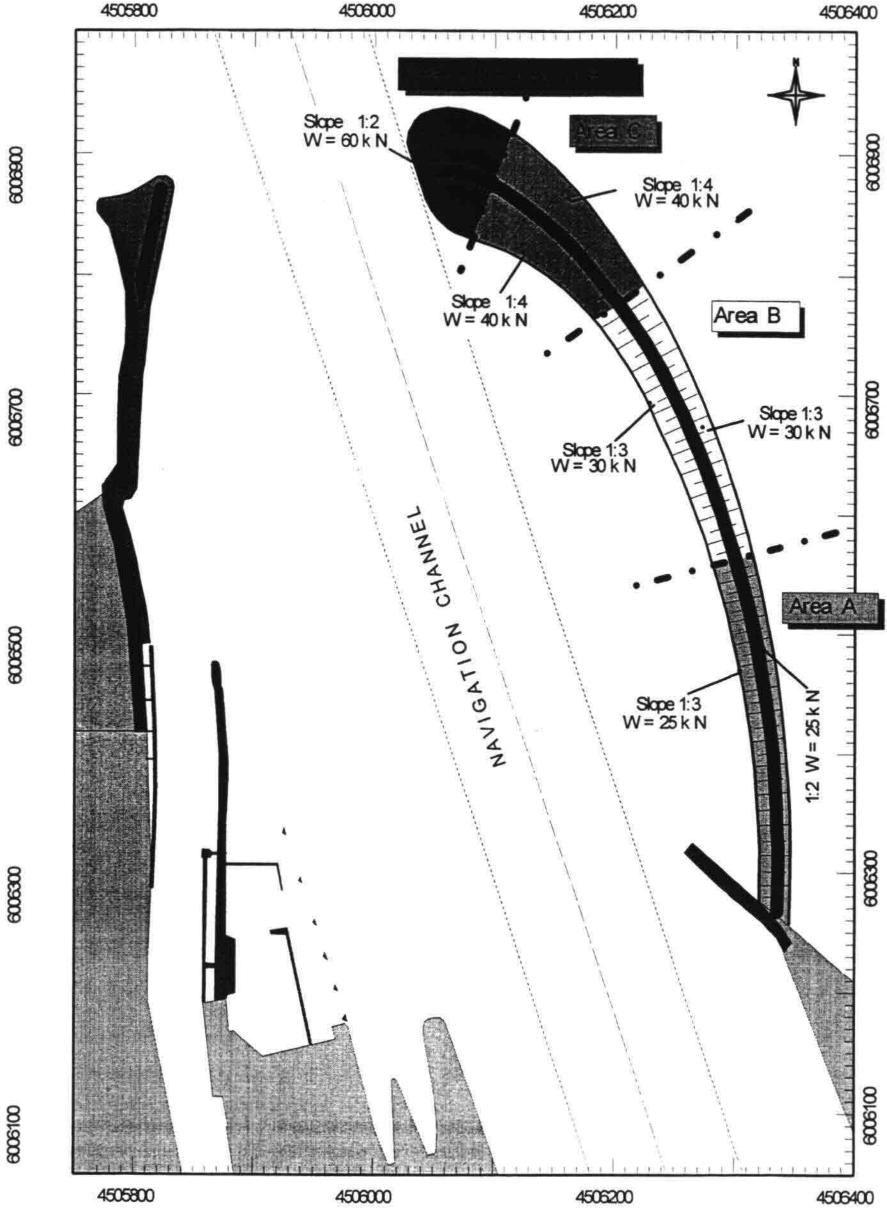


Fig. 12: Design of new breakwater system at Warnemünde / Rostock

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- Technical Excursion

Thursday, September 11th
Boat trip (Harbour of Rostock): Vessel 'Tonnenleger Buk' of WSA Stralsund
Construction site of the New Breakwaters Warnemuende
Information by WSA Stralsund and ARGE Molenbau
Tour around the Port of Rostock
Information by Seehafen Rostock Verwaltungsgesellschaft mbH
Coastal Protection Works at Fischland, Darss and Zingst
Information by StAUN Rostock

Friday, September 12th
Seaside Resort Prora
Ferry-Port Mukran
Viewpoint: Stubbenkammer, Isle of Rügen

Saturday, September 13th
Historic and Cultural Tour Berlin

Sunday, September 14th
Guided City Tour
Visit to Potsdam Palace (Schloß Sanssouci)

Monday, September 15th
Waterway - Crossing of River Elbe and Midland Channel
(Projekt 17 Deutsche Einheit):
Ship Elevator Magdeburg Rothensee and
Construction Site New Lock Rothensee
Information by WNA Magdeburg and WSA Magdeburg
Large Wave Channel (GWK), Hannover
Information by Forschungszentrum Küste (FZK)

Tuesday, September 16th
Departure of Chinese Delegates