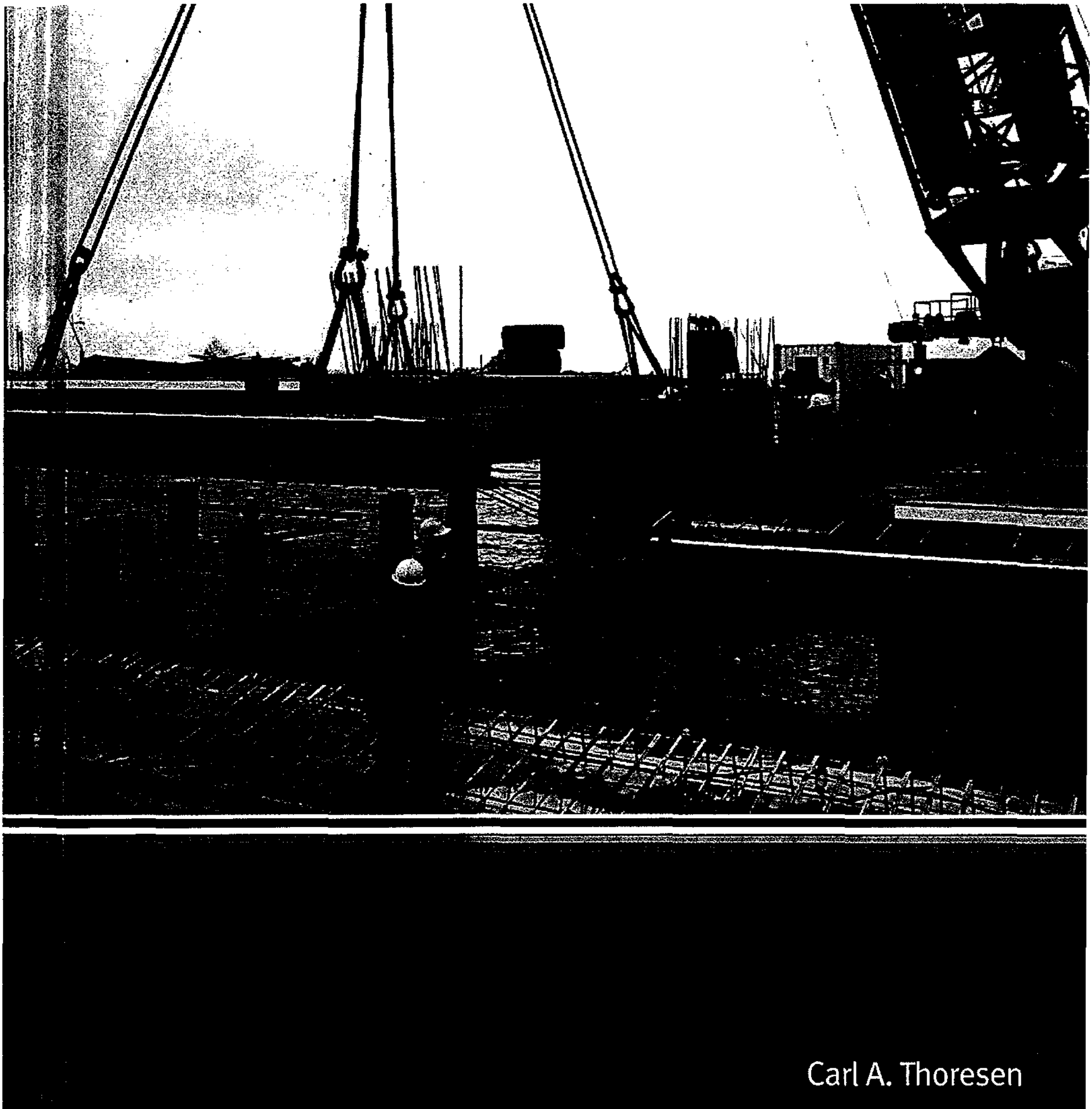
PORT DESIGNER'S HANDBOOK: RECOMMENDATIONS AND GUIDELINES



Carl A. Thoresen

Port designer's handbook

Recommendations and guidelines

Carl A Thoresen

ThomasTelford

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If you copy or steal from one,
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it is called research.

Wilson Mizner

This book is dedicated to my wife, Liv. Thanks to her understanding and encouragement I was able to do my research. Without her I would neither have been able to start nor finish this book.

Carl A. Thoresen

Contents

Foreword

The invitation to write the foreword to *Port Designer's Handbook* gives me the ideal opportunity to express my belief that harbour experts should write and share their knowledge and information more often to their colleagues, and thereby contribute and make available more literature and information on their experience and knowledge to others within the harbour and port sector.

This book will, in my opinion, give harbour colleagues around the world an opportunity to study Norwegian practices and solutions in the design and construction of ports, from traditional berth structures to complicated oil and gas berths.

This handbook gives valuable information and experience to practising harbour and port engineers, postgraduate and senior university students.

Øyvind Stene
Director General of the Norwegian Coastal Administration

Preface and acknowledgements

Over the past ten to twenty years there has been considerable improvement and new thinking in the design and construction of port and harbour structures. The purpose of this handbook is not to explain the full scope of port design and construction but to give guidelines and recommendations, and to try to deal with some of the main items and assumptions in the layout, design and construction of modern port structures and the forces and loadings acting on them. The use of concrete for berth structures in a marine environment is also dealt with in detail, as well as the types of deterioration and methods of repair of these structures.

This handbook is mainly based on the author's nearly 40 years of experience gained from practical engineering and research from more than 500 different port and harbour projects, both in Norway and other countries, as Chief Port and Harbour Engineer and technically responsible for the port and harbour division in the largest Norwegian consulting company, Norconsult AS. This handbook also includes material from many lectures, held over the years, by the author on berth and port structures at the Norwegian University of Science and Technology, Norway, and at the Chalmers University of Technology, Sweden, and on postgraduate courses arranged by the Norwegian Society of Chartered Engineers.

Over the last twenty years or so, the growing interest in the design of port and harbour structures has produced a huge amount of research reports, technical papers and information, especially from the International Navigation Association, PIANC. Much of the information has been evaluated and summarized in this handbook.

The author hopes that the contents will make a readable and useful handbook and a practical guide for port engineers who are responsible for design of port and harbour structures, and that the handbook will contribute to further development of the subject. Many of the subjects mentioned in the text are worth a further study, which the reader can research by referring to the Further Reading list at the end of each chapter.

Finally, the author would like to express his deepest thanks to the many friends and colleagues who have contributed with helpful encouragement, information, comments and suggestions. A special and sincere thanks goes to the following persons, to whom the author has drawn upon for their experience and knowledge, and therefore deserve to be mentioned specially for their contribution to the book:

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A special thanks to Thomas Telford Publishing for all their help and assistance while I was writing my manuscript.

About the author

Carl A. Thoresen graduated from the University of Strathclyde, Glasgow, Scotland in 1963. He has, during his professional career worked, for the largest Norwegian consulting company, Norconsult AS, as Chief Engineer and has been the consultant responsible for more than 500 different port and harbour projects in Norway and abroad.

The projects have included waterways, lighthouses, breakwaters, fishing harbours, marinas and small-craft harbours, ferry berths, multi-purpose and container terminals and commercial ports, breasting and mooring dolphins and complicated oil and gas harbours. The works have taken in planning and technical and economical evaluations, design, preparations of tender documents, tender evaluations and negotiation, construction supervision, maintenance and rehabilitation work.

He is author of the book *Port Design. Guidelines and Recommendations* (published 1988), co-author of the PIANC publication *Development of Modern Marine Terminals*, co-author of the PIANC publication *Fender System 2002*, co-author of the *Norwegian Recommendations for Waterfront Structures* and co-author of the *Norwegian Recommendations for Concrete Construction in Water*. He has written about 50 papers and articles on port and harbour planning, berth and fender design and constructions and marine repair and rehabilitation works.

He has, over the years, held lectures on berth and port structures at the Norwegian University of Science and Technology, Norway, the Chalmers University of Technology, Sweden and for post-graduate courses arranged by the Norwegian Society of Chartered Engineers. He has been actively engaged in international conferences and has been a member of many official technical committees for harbour design, constructions and waterfront structures.

1

Port planning

1.1 Introduction

The advantages and disadvantages of various berth alternatives for accommodating all types of ships in a port cannot be assessed in detail without well-developed and well-defined port plans. All port plans represent a set of compromises between several goals. In this chapter an evaluation of the activities required for the preparation of a detailed port plan is given, discussing the criteria which are needed as a basis for the planning from the open sea, through the approach channel, the harbour basin, the berth and the terminal as indicated in Fig. 1.1.

The port authority and its Consulting Engineers should identify the activities required to be able to establish the Terms of Reference for the engineering planning, and to specify the work to be executed by, for example, the Consulting Engineers, the contractors, the port operators, etc. within the fixed margin of expenditure.

1.2 Planning procedures

There are many activities, which have to be recorded, clarified and assessed. Essential basic information includes, inter alia, data on the physical and technical conditions in the development area and information from experienced port users. A checklist for the planning of port developments should at least cover the following main items:

(a) resolution by the port authority to start planning
(b) selection of Consulting Engineers

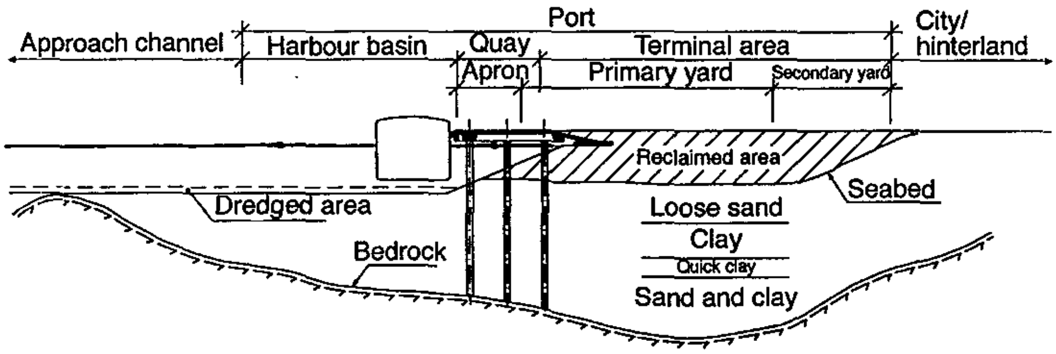

Fig. 1.1. The activities necessary to be investigated

(c) scope of work:
  (i) introduction
  (ii) background
  (iii) scope of project
  (iv) basic data
(d) registration of users:
  (i) public
  (ii) private
(e) recording of users' needs:
  (i) types of port and berth structures
  (ii) traffic statistics
  (iii) types and specifications of ships
  (iv) coastal areas and maritime conditions
  (v) berth and land area requirements
  (vi) growth factors
(f) impact study
(g) site evaluation:
  (i) existing areas
  (ii) potential areas
  (iii) natural conditions
  (iv) relationship with neighbours
(h) layout plan
(i) economic analysis
(j) work schedule.

The above-mentioned items will be outlined in the following sections, in order to describe the various activities which require closer study and assessments in connection with proper port planning. But one shall always remember that ports often define their own needs. Some ports are predominantly bulk ports, others high-value cargo ports and others multi-purpose ports, etc. Based on the character of existing traffic and expectations about future potential, the port needs and future capacities will vary. A port usually exists in a dynamic business and social environment, and therefore the needs of the port change rapidly over short periods of time.

1.2.1 Resolution to start planning

After engaging a consultant, but before the planning starts, it is essential that the Client or the authority concerned has prepared a project plan stating clearly the conditions and target of the planning or the work to be done.

The planning and implementation of a project for a new port or for a major port extension can be subdivided into the following main phases:

(a) project identification study
(b) preliminary planning study:
  (i) reconnaissance mission
  (ii) fact-finding mission
  (iii) feasibility study
  (iv) appraisal mission and study
(c) detail planning work:
  (i) inception planning
  (ii) interim planning
  (iii) final planning and report
(d) pre-engineering work:
  (i) design criteria and structural specifications
  (ii) preliminary cost estimate
  (iii) final pre-engineering report
(e) detail engineering work:
  (i) design calculation
  (ii) tender drawings (formwork drawings)
  (iii) technical specification for construction
  (iv) bill of quantity
  (v) tender evaluation
(f) construction work:
  (i) construction drawings
  (ii) construction supervision
(g) project completion report.

One of the most important tasks is probably the preliminary planning study, which serves the purpose of verifying whether a suggested project is really justified from an economic point of view and whether it can be implemented at a reasonable cost under safe technical conditions. The most convenient site for the suggested works should be tentatively selected, or alternative locations suggested. A preliminary plan of the port, an approximate cost estimate and an economic and financial evaluation should form the final part of a preliminary study report. The results of the planning study on port development should therefore always be summarized into an action-oriented programme containing an evaluation of the following:

(a) operational analysis
(b) technical analysis
(c) economical analysis
(d) financial analysis.

The fate of the project will, therefore, depend on the conclusions of the preliminary report. The general character of the port, the layout of the port facilities, their capacity and extent are determined by the preliminary plan, notwithstanding such modifications or corrections that may be made afterwards. The preparation of a preliminary study should, therefore, be entrusted to port planners with the widest possible range of experience, both in technical planning and in port operation under various conditions, and with a thorough understanding of economic and transportation problems. When the general conclusions of the preliminary study have been approved and its recommendations accepted, the next predominantly technical phase of the planning will include all necessary field investigations and the detailed design.

1.2.2 Selection of Consulting Engineers (planners)

It is a fact of life that the competition for consultancy business in the harbour sector is now tougher than it has ever been. It is therefore necessary for consultancy companies to be highly specialized in the use of the latest technical skills and development tools. The company's past experience and performance as a whole, the experience of the leading personnel who will be involved in the project and the company's proposed methodology are important factors for the Client to consider when selecting a Consulting Engineer. As a basis for selection, the Consulting Engineers must enumerate and describe the projects they have undertaken, naming previous employers for reference. They should also indicate the general manpower available (e.g. graduate engineers), whether they can step-up planning and design if so desired by the Client and whether they can mobilize divers (frogmen), underwater camera, diver's telephone outfit, etc., to carry out underwater investigations and supervision.

A Client should always make sure that the personnel named in a proposal would also form the project team working on the project. If a team member should be replaced, the Client will always demand that the new team member should have at least the same qualifications.

The Fédération Internationale des Ingénieurs-Conseils, FIDIC, has the following policy statement:

> A Consulting Engineer provides a professional service. A Client, in selecting a Consulting Engineer, is selecting a professional adviser. The Consulting Engineer's role is to put expert knowledge at the disposal of his Client. On engineering matters, he serves his Client's interests as if they were his own. It is essential that he should have the necessary ability. It is equally important that the Client and Consulting Engineer should proceed on the basis of mutual trust and co-operation. In the professional relationship, the Consulting Engineer identifies with his Client's aims.

It is in the Client's interest to select the most qualified and experienced company and to negotiate a fair price for the consultancy services. Saving one or two per cent of the project cost on engineering is penny-wise and pound-foolish.

Payment for consultancy services can be defined in the following ways:

(a) payment on a time basis
(b) payment of a lump sum, based on either:
  (i) the Consulting Engineer's estimate of the work involved, on
  (ii) a generally accepted fee scale
(c) payment as a percentage of the cost of the Works.

Direct expenses, such as travel costs, hotels, etc., are normally reimbursed separately. The fee for the consultancy service itself is normally invoiced at agreed intervals.

Some of the international development banks select the Consulting Engineer after what was previously called 'The Two Envelope System'. The Consulting Engineer is requested to submit his proposal for consulting services in one technical envelope and one financial envelope. The technical proposal should be placed in a sealed envelope clearly marked 'Technical Proposal' and the financial proposal should be placed in a sealed envelope clearly marked 'Financial Proposal'.

The **technical proposal** should contain the following:

(a) The consulting company's general expertise for doing the work. If the company do not have the full expertise, the company can be associated with another company. For work in a developing country it is considered desirable to associate with a local company.
(b) Any comments or suggestions on the Terms of Reference (TORs) and a description of the methodology (work plan) the Consulting Engineer proposes executing the services, illustrated with bar charts of activities, and the graphics of the type of Critical Path Method (CPM) or Programme Evaluation Review Technique (PERT).
(c) The estimated number of key professional staff required to execute the work according to the TORs. The composition of the proposed engineering team, and the task which would be assigned to each member. Estimates of the total time effort supported by bar chart diagrams showing the time proposed for each team member.
(d) Curricula vitae (CVs) for the proposed key team members. The majority of the team members should be permanent employees of the company.
(e) If the Terms of Reference specify training as a major component of the assignment, the proposal should include a detailed description of the proposed methodology, staffing, budget and monitoring.

The maximum of 100 points given to the technical evaluation is usually divided as follows:

(a) Specified experience of the company related to the assignment: 10 points.
(b) Adequacy of the proposed work plan and methodology in responding to the TORs: 30 points.
(c) Qualifications and competence of the consulting team: 60 points.

The **financial proposal** should contain and list the cost associated with the assignment as indicated in the following:

(a) The remuneration for the staff assigned to the team, either foreign or local, in the field and at headquarters.
(b) Subsistence per diem, housing, etc.
(c) Transportations for the team — both international and local.
(d) Mobilization and demobilization.
(e) Services and equipment such as vehicles, office equipment, furniture, printing, etc.
(f) The financial proposal should take into account the tax liability and cost of insurances.

The **evaluation of the proposals** will usually be carried out according to the alternatives described below:

(a) In the case of the 'Fixed Budget Selection', the Client and/or the Evaluation Committee will select the firm that submitted the highest ranked technical proposal within the indicated budget (normally given with the invitation). Proposals that exceed the indicated budget or which do not get the minimum score, usually 75 out of 100 points, will be rejected. The company having the best technical proposal will then be invited to contract negotiations.
(b) In the case of the 'Quality-based Selection', the highest ranked firm on the basis of both the technical and the financial proposal is invited to negotiate a contract. Under this procedure, the lowest financial proposal will be given a financial score of 100 points. In this case the financial proposal will typically count as 20 per cent whereas the technical proposal will count as 80 per cent in order to reach the overall score.

1.2.3 Scope of work

The Consulting Engineers should assist the Client in defining the assignment. The following should therefore be clearly specified.

Introduction

(a) Client and Client's contact persons
(b) type of project (preliminary or final design)
(c) project's geographical position and boundaries
(d) who would the project affect (people, firms, etc.)?

Background

(a) Project background (existing infrastructure, traffic increase, development restrictions, old installations, excessive maintenance costs at existing facilities, etc.).

Scope of project

(a) Project area and boundaries
(b) project involvement (activities, nature of work, scope of project, etc.)
(c) project's execution schedule.

Basic data

(a) Which reports and data can be used as a basis? When were the reports prepared and by whom?

1.2.4 Registration of users

Experience has shown that it can be difficult to register all port users who may influence the preparation of a port plan, i.e. present and potential users of the harbour facilities. It is advisable to register users in the following two groups.

Public users

(a) Port authorities, municipality, district, state, etc.

Private users

(a) Shipping companies
(b) private industry, service industry (charterer, stevedores, etc.)
(c) clubs (marinas, etc.).

1.2.5 Recording of users' needs

The rapid growth of regional and international trade has placed increasing demands on the shipping and port sectors. Therefore, in order to achieve a realistic port plan it is essential to record the needs of users which may have a bearing on the plan and to obtain relevant data on this subject. The recording of users' requirements must be carried out in close cooperation with the Client or port authority. This also includes organization of the port itself. The following are different approaches to organizing the port:

(a) **Resource (tool) port**: the port owns the land, infrastructure and fixed equipment, provides common-user berths and rent-out equipment and space on a short-term basis to cargo-handling companies and commercial operators.
(b) **Operating (service) port**: the port provides berths, infrastructure and equipment together with services to ships and their cargo.
(c) **Landlord port**: for larger ports this is the most common system where the port owns the land and basic infrastructure and allows the private sector to lease out berths and terminal areas.

In order to obtain a general view of the users' needs the recording should be carried out as outlined below.

Type of port facilities

The type of port facilities should be evaluated and registered as follows:

(a) Evaluate and register the various types of port facilities that exist and identify those that would meet the future requirements (commercial port, bulk-cargo port, industrial port, fishing port, supply port, ferry berths, marinas).
(b) Register which of the facilities would be public, or privately owned.

Recording of traffic data cargo volumes

The planner should have access to statistics compiled either by private companies or by port authorities on traffic, density and volume/tonnage of goods handled in the harbour area. Unfortunately, the statistics seldom specify the type, size, weight and other details of individual consignments. It may therefore be necessary to carry out additional recording and research for a limited period of time in order to obtain annual, weekly and daily averages of the port traffic and to identify peaks. This recording, or the more detailed research, must be oriented towards the port plan objectives. The general pattern of the cargo flow which can be expected through the terminal area of a port, is illustrated in Fig. 1.2.

The recording of traffic densities and cargo volumes should give a detailed account of cargo and passenger handling by day of week, hour of day, and mode of transportation to and from the port, for the following:

(a) ocean-going tramp ships
(b) foreign liner ships
(c) domestic liner ships
(d) ferries
(e) trucks
(f) buses
(g) railways
(h) possibly aircraft.

The annual turnover in the port should, if possible, be subdivided into the following categories:

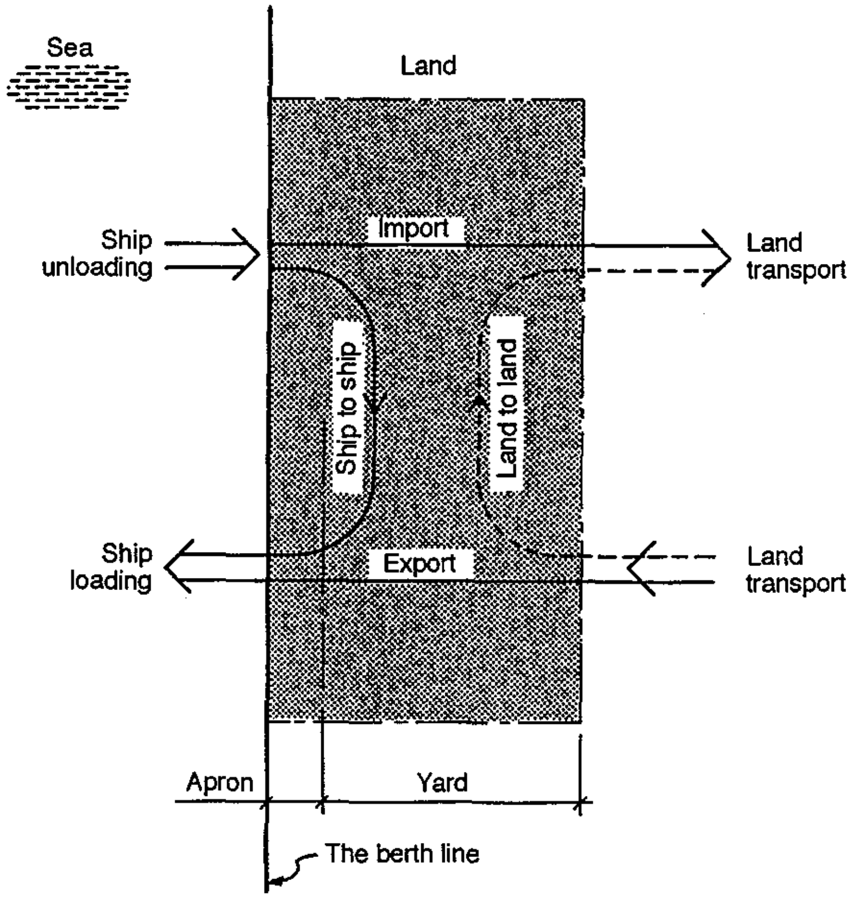

Fig. 1.2. Cargo flow through a port

(a) bulk/general cargo
(b) transhipment ship/ship
(c) transhipment ship/rail
(d) goods carried by coastal ships/tramps
(e) goods/general cargo handled at terminal
(f) storage time
(g) type of storage
(h) customs clearance.

Commodities should be described in detail:

(a) type of cargo
(b) present and potential cargo tonnage and volume
(c) frequency of cargo arrival
(d) origin and destination of cargo
(e) times of loading and discharging
(f) space requirements for cargo
(g) cargo-handling rate/time of storage

(h) commodity classification
(i) cargo-handling operations analysis
(j) storage requirements (cold or warm).

It is essential to specify if the goods require special handling equipment, such as

(a) loading and unloading equipment
(b) capacities of cranes (mobile or stationary)
(c) fork-lift truck requirement.

Based on the above-mentioned data it is possible to evaluate the optimum storage time. Generally owners prefer the goods to remain in harbour until required. This means that, for the owner, unnecessary loading/unloading should be avoided. On the other hand, port authorities want goods to leave the harbour as quickly as possible so as to clear the harbour for new arrivals. Therefore, the design criteria to be adopted for the harbour area should be based on actual shipping statistics and cargo volumes.

For the majority of ports, the storage time in the port area is one of the most critical factors in evaluating the port capacity. Nowadays, with faster loading and unloading of ships, it is very seldom the berthing capacity (the number of berths) that reduces the port efficiency. As a rule, it is the limitation of the storage area behind the berth that is the determining factor. Roughly, one can say that if the storage time can be halved the harbour capacity can be doubled. This would ensure the best use of invested capital and would, at the same time, result in lower cost/ton handled.

Types and specifications of ships

During the past 20–30 years, the trends in shipping have had a great impact on the port and harbour development. The larger tankers, container ships and cargo ships require deeper water and highly mechanized cargo-handling equipment and systems. The rapid growth of, for example, containerization has had a great effect on the handling equipment, the layout of the yard and the size of the berth. For this reason the following must be studied closely:

(a) ship types (fishing, cargo, roll on/roll off (ro/ro), load on/load off (lo/lo), tankers, warships, etc.)
(b) ships' sizes
(c) frequency of arrivals and times of day

(d) ships' origins and destinations
(e) analysis of future conditions.

Based on ship parameters, one can analyse the demand for berth facilities and determine the required water depths at the various berth structures.

Coastal areas and maritime conditions

Port basin requirements, as well as local maritime conditions, have sometimes been neglected in port planning. A port plan should include information about the following:

(a) general conditions concerning navigation between open sea and berth facilities, tugs required, anchorage grounds, waiting area
(b) length, width and depth of access channel and basin area, and depths alongside
(c) recording of submarine cables, etc.
(d) restrictions in manoeuvring conditions due to wind and current and possibly in waiting time for better weather conditions
(e) need for shelter
(f) requirements regarding pilot services, beacons, safety zones, tug-boat assistance, etc.
(g) collision possibilities and other dangers such as height obstructions (bridges, high-voltage lines)
(h) possible restrictions regarding berthing and departure times.

Berth and land area requirements

Port owners should be aware of the danger of not forward planning and they should always be conscious of possible future requirements. Therefore, one of the most important items of a long-term port plan should be flexibility. A good plan is one in which the basic strategy remains intact even when some of the details of the port plan need to be adjusted.

In addition, probably the most important function of the port plan is to reserve land for the port to develop and to expand to meet future growth in traffic or changes in technology. Therefore the following information should be obtained:

(a) location of the port area in relation to:
  (i) regional conditions

- (ii) local conditions
- (iii) local traffic (railways, trucks, bicycles, passenger traffic, pedestrians, ferry traffic with parking space for waiting vehicles, etc.)
- (iv) car parks (public and private).

(b) Location of berths:

- (i) general requirements
- (ii) relationship to port area
- (iii) natural conditions
- (iv) neighbourhood relationship.

(c) Size of berth:

- (i) types and numbers of berths (general cargo, containers, ro/ro, lo/lo, bulk, etc.)
- (ii) length and surface of each berth, depth alongside
- (iii) dolphins
- (iv) special mooring facilities
- (v) loads
- (vi) utilization of the port facilities
- (vii) utilization of berth capacities for separate berths.

(d) Land area:

- (i) present and future needs for land areas
- (ii) indoor and outdoor storage capacity
- (iii) access roads
- (iv) development of land area (gravel, asphalt, type of traffic, etc.)
- (v) area restrictions (building lines, cables, power lines, etc.)
- (vi) facilities for dockers, stevedores, service functions
- (vii) local authorities' demands and benefits.

(e) Demand for auxiliary services and installations:

- (i) electricity, water and telephone connections
- (ii) lighting
- (iii) mooring facilities, fenders
- (iv) life-saving equipment and ladders
- (v) refuse collection and disposal, cleaning of tanks, waste-water tanks, oil-protection and fire-fighting equipment
- (vi) water and fuel bunkering
- (vii) maintenance facilities
- (viii) repair workshop, slip.

When evaluating potential sites, it is advisable to divide the port into activity zones.

1.2.6 Growth factors

During the recording of available data to be used in the port planning, the planner must bear in mind the consequences a port development will have on the society:

(a) population increase locally and regionally
(b) economic growth
(c) traffic growth and modified transport modes
(d) industrial developments
(e) environmental problems.

1.2.7 Impact study

In recording users' requirements, the planner must assess whether the existing activities in the harbour can, or have to, be relocated and what the impact will be for a specific user.

Would such relocation represent great expenses to the user and thus make him less competitive? Or would the relocation to more developed areas make the user more competitive?

At an early stage one must assess the negative impacts that may arise. For instance, increased ship traffic would also increase the possibility of ships colliding. This could be disastrous, especially if ships carrying dangerous cargo, such as gas, ammunition, etc., were to collide. In other words, an impact study must consider all risks involved.

1.2.8 Site evaluation

Potential sites for harbour development must be examined to ensure that they meet the specific functions of the port and the needs of various port users. A detailed investigation is often not necessary, but the examination should provide sufficient information for the evaluation. The following items should be considered.

Natural conditions

It is often difficult to visualize the effect the natural conditions may have, but problems can be solved by means of model studies. The following must be evaluated:

(a) Topographical and maritime conditions:
   - (i) description of land area/topographical conditions
   - (ii) hydrographical conditions
   - (iii) manoeuvring and navigation conditions.

(b) Geotechnical conditions:
   (i) stability and load-bearing capacity
   (ii) choice of type of berth structure
   (iii) location of berth based on geotechnical evaluation
   (iv) seabed conditions
   (v) dredging and blasting conditions
   (vi) dumping authorization
   (vii) sounding, acoustic profiles.

(c) Geological conditions:
   (i) structure and composition of strata.

(d) Water-level recordings:
   (i) tidal variations
   (ii) depth references.

(e) Water quality:
   (i) quality of water (pH value, salt content, etc.)
   (ii) degree of pollution
   (iii) visibility
   (iv) corrosion characteristics (corrosion, deterioration of concrete, attacks by marine borers).

(f) Wind:
   (i) wind force, directions and durations (compass card)
   (ii) critical wind forces and directions.

(g) Waves:
   (i) wave heights caused by wind, significant wave length, maximum wave height, wave direction
   (ii) swell
   (iii) waves from passing vessels.

(h) Climatic conditions:
   (i) air temperatures (maximum, minimum)
   (ii) air humidity
   (iii) water temperature.

(i) Current:
   (i) strength, direction and duration
   (ii) erosion and siltation, sea-bottom conditions.

(j) Ice:
   (i) thickness, duration, extent
   (ii) possibility of ice-breaking assistance.

(k) Visibility conditions:
   (i) fog and number of foggy days
   (ii) topographic conditions
   (iii) need for navigational aids, lighthouse, radar, radio.

(l) Evaluation of natural resources:
  (i) impact of development on environment.
(m) Model testing:
  (i) stability, protection measures and facilities
  (ii) erosion and sedimentation frequencies.
(n) Materials:
  (i) investigation of available local construction materials.
(o) Contractor's equipment:
  (i) assess type, availability and capacity of equipment for marine structures.

The choice of type of berth structure and layout should be based on thorough knowledge of natural conditions and market circumstances. Superficial investigations of these factors could result in severe negative economic consequences.

Relationship with neighbours

The following should be assessed to determine the impact of a port plan on the neighbourhood:

(a) Existing properties:
  (i) recording of private ownership of land, of port installations and plants which would be affected by the port development
  (ii) power lines and cables in the area, water supply and sewerage systems, etc.
(b) Local traffic:
  (i) by land
  (ii) by sea (size of ships and frequency of calls)
  (iii) hindrance and delay caused by traffic from neighbours
  (iv) anchorage grounds required by neighbours
  (v) manoeuvring needs of neighbours
  (vi) tugboat use by neighbours.
(c) Cargoes:
  (i) existing and future volumes and tonnages of cargo to be transported to adjacent premises by land or sea.
(d) Offshore traffic:
  (i) ship traffic outside the development area, size of ships and frequency, range and sailing channel.
(e) Future traffic:
  (i) forecast of sea borne and overland transport in the area

   (ii) frequency of calls at neighbour berth and movement of vessels offshore.

(f) Damages and drawbacks:
   (i) damages, drawbacks and consequences to neighbours due to their own development and traffic
   (ii) damages and drawbacks caused by a third party's development (waves and nuisance due to increased traffic)
   (iii) traffic conditions (queues, etc.).

(g) Possibilities of expansion:
   (i) acquisition of neighbouring sea and land areas
   (ii) acquisition costs.

1.2.9 Layout plan

Based on records of users, needs of users, assessment of consequences, location of site, etc., a port layout plan can be prepared to cater for the various activities in the area. Such a layout plan can, in addition to selecting the most convenient technical solutions, also include various political solutions, which will not be dealt with here.

1.2.10 Economic analysis

All port projects can be broadly divided into the following three main groups:

(a) rehabilitation of existing port facilities
(b) expansion of port facilities
(c) development of a new port.

Therefore, in order to obtain a picture of what a project would cost, it is advisable to show the costs of the planning phase and of the development phase separately:

(a) The planning phase expenditures will depend on how much the Client (port authorities, municipalities, government, etc.) is prepared to invest in planning.

   In order to assess the expenditures involved in the planning, one must know how much data collection, site investigations, etc. will or can be carried out by the port authorities, municipal engineers, port users, etc. To obtain a realistic design, geotechnical evaluations of the areas concerned should be carried out. This would facilitate the allocation of areas to the various activities.

(b) The construction phase expenditures will mainly comprise the construction expenses plus the Consulting Engineers' study expenses. The latter usually amount to 3–5 per cent of the construction expenses.

In connection with planning there will almost always be alternative development options, which can influence the choice of berth structure. This requires an adequate knowledge of engineering techniques and an understanding of the requirements to ensure a satisfactory cost–benefit ratio for the project.

(c) There should also always be competitive tendering between the construction contractors if possible, because without competition there can be no true cost comparison. No competition inevitably leads to a low level of efficiency, and political decision makers may be unable to see possibilities to save on investment or operating costs, even when these possibilities are within reach.

In principle, the port development should earn a satisfactory financial rate of return, but this could sometimes be difficult to achieve during the first years after the completion of the development, due to the fact that it might take some years for the port traffic to build up. In the development of a new port, some construction items, such as a new breakwater, large dredging works, etc., will have long service lives and might be adequate for further port expansion in 20 or 30 years. Therefore, during the first years of operation it may not be possible for the traffic of a new port to be strictly cost-based.

1.2.11 Work schedule

A complete port plan includes a programme for a staged development of the port. One must first of all try to record when the respective parts of the port should be made ready for the respective users. This work schedule will also indicate when construction work should start on the various new areas.

1.3 Subsurface investigations

1.3.1 General

One of the most important factors in the planning and design process is to get detailed knowledge of the geotechnical and the subsurface conditions of the port area before a proper development of a port project. This is an important basis for selection of the most suitable port structures, efficient design of the facilities, and a smooth construction process. Construction difficulties and cost overruns of such projects are most commonly caused by unexpected soil or rock conditions. Sufficient site investigations can therefore be considered as a relatively cheap 'insurance premium' to reduce the risks of encountering technical and economical problems during implementation.

The investigations shall provide data for various engineering and construction issues, such as:

(a) foundation of onshore and marine structures
(b) settlement of reclaimed areas
(c) stability of filled and dredged slopes
(d) nature of soils to be dredged and suitable dredging methods
(e) usage of dredged materials
(f) sources and properties of natural construction materials, such as embankment fills, aggregates, pavement materials, armour stones, etc.

The necessary extent of the investigations and depth of boreholes depend on the type and size of project, the complexity of the site conditions and the information that is already available. Investigation depth should be deeper than any potential stability failure surface, and deeper than the possible penetration of foundation piles or sheet piles. British Standard BS 6349-1 (Article 49.6) recommends a boring depth below base of the structure of a minimum 1.5 times the structure width, and in dredging areas to hard strata or a minimum 5 m below dredging level.

The first step in the search for data of the soil and rock conditions is collection of existing information. The main sources are:

(a) geological information from maps, reports and study of aerial photos
(b) reports from earlier physical investigations in the area, collected from official or private files
(c) previous history and use of the site including experience from construction works in the vicinity and any defects and failures attributable to ground conditions.

The next step should be to analyse the project and the need for additional information, in order to establish a programme for necessary site investigations. For large projects, such investigations will normally be done in steps corresponding to the requirements of the different planning stages, and the detailed and most costly investigations are referred to the design and construction phases. Sufficient data should, however, be available before a construction contract is signed.

The port authority or Client and his consultant normally determine the necessary extent and types of investigations but there may also be minimum requirements stated in local codes, such as building codes in the USA.

1.3.2 Organization of the site investigations

Methods and procedures for undertaking site investigations vary from one country to another, depending on local practice and available resources. It is generally advisable to follow local practice if it has proved successful in the past.

In some countries or areas the authorities have their own equipment for basic investigations. The normal procedure is that the employer or Consulting Engineer engages a firm to carry out the site investigations. Such firms could be consultants or contractors with a range of investigation equipment, or specialist firms with a narrow product line, for instance, geophysical investigations.

Offshore investigation can be performed from a barge properly anchored and positioned. There are also available vessels and jack-up platforms, specially equipped for subsurface investigations, which are used on large projects or at great water depths. Survey for positioning is usually done using a global positioning system (GPS).

A normal procurement procedure is that qualified firms are invited for competitive tenders. A contract is established between the Client or engineer and the successful firm(s) based on recognized local or international contract conditions, for instance the 'FIDIC Model Subconsultancy Agreement' or for large assignments the FIDIC Red Book.

The necessary extent of supervision by the Client or engineer will depend on the reliability of the selected firm, but some monitoring and supervision of the investigations are highly recommended. Field investigations and laboratory testing should be done according to recognized procedures, which for most methods are stated in international standards or codes of practice, such as the American Society of Testing Materials (ASTM) and American Association of State Highway Officials (AASHTO) standards, British standards or codes (BS) or European EU standards.

The contractor is normally required to submit progress records and boring logs during the course of the work and a final report with documentation of all investigation results.

The available investigation methods may be divided into the following types or groups:

(a) geophysical methods
(b) soundings or simple borings
(c) in situ tests
(d) soil and rock sampling
(e) field trials
(f) laboratory tests.

The methods most commonly used are described in the following sections. Different methods may provide similar information. The selection of method will, to a large extent, be determined by availability and costs. Each method has, however, certain advantages and limitations which should be taken into consideration.

1.3.3 Geophysical methods

Geophysical methods are suitable for initial surveys offshore because they can provide a significant amount of information at a modest cost. Such methods do not provide accurate data on soil and rock conditions and need to be calibrated by information from borings, sampling and/or in situ tests.

The commonly used methods are seismic reflection and seismic refraction.

Seismic reflection (acoustic profiling)

Acoustic profiling is performed by equipment towed by a vessel at a low speed (2–5 knots); working according to the same principle as an echo sounder used for depth recording, but operating at a different frequency which allows penetration into the seabed. A high-energy acoustic pulse passes through the water and into the soil, as shown in Fig. 1.3. It is reflected back by the seabed and by subsoil interfaces corresponding to changes in density and sonic velocity. A profile of the return times from the various reflecting horizons is continuously recorded and transformed to electric signals that are visualized by printing in a two-dimensional diagram (echogram), which has to be interpreted by a specialist.

Different types of equipment are available, each of which has different characteristics and capabilities. The main difference is the frequency, which determines the maximum penetration depth and

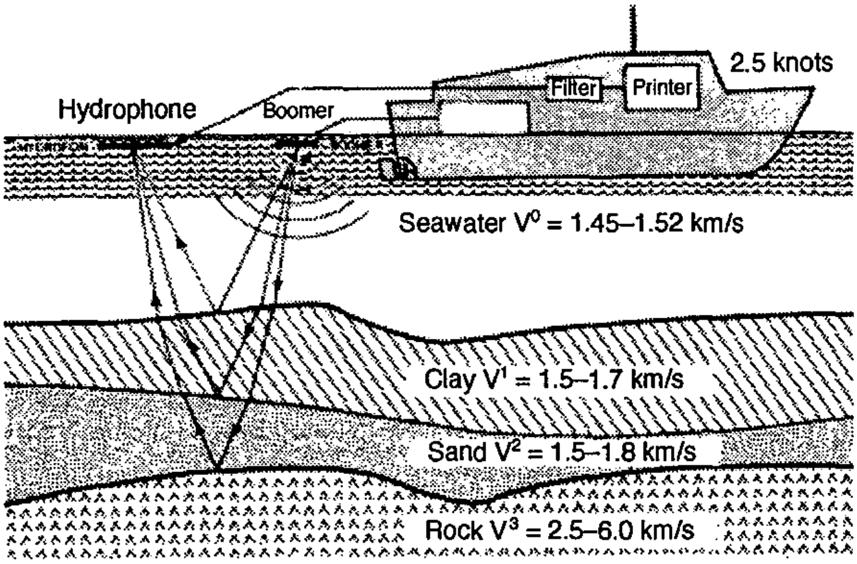

Fig. 1.3. Acoustic survey

the resolution accuracy. Low-frequency pulses achieve deep penetration but have a low resolution. The main types are:

(a) Penetrating echo sounder (pinger): frequency 3–10 kHz, penetration depth up to 25 m, and resolution around 0.5 m.
(b) Boomer, consisting of a metal plate placed on a small catamaran: frequency 0.5–2.5 kHz, penetration depth 100 m, resolution 1 m.
(c) Sparker, consisting of a metal frame with electrodes, which is towed slightly below the water level: frequency 0.05–1.0 kHz, penetration depth 500 m, resolution 5 m.

The indicated penetration depths are valid for sediments of clay, silt and sand. The results achieved from such a survey are the thickness of soil above bedrock and borders between different soil types. Determination of soil layering is uncertain unless there are large and distinct changes in the seismic velocities.

The main advantage of the method is that a rough picture of the subsoil conditions in large areas can be obtained fast and at a modest cost. With measured profiles at a distance of 10 m, an area of around 20 km^2 may be covered in one day. The disadvantage is that the results are not accurate and need to be checked and calibrated by boring and or sampling.

Seismic refraction

Seismic refraction survey can be used on land and in the sea. A system of seismometers (geophones) are placed along a profile and connected

Fig. 1.4. Difference between the reflection and the refraction method

by cables to a recorder. A shock wave is initiated by explosives or by a hammer blow. The wave travels faster in more consolidated soil than in soft soil, and is fastest in compact bedrock. By recording the travel time of the direct and reflected waves to the geophones, a profile of soil boundaries and depth to bedrock can be obtained.

Seismic refraction is more accurate than reflection survey, as shown in Fig. 1.4, but the capacity is less and it is therefore more costly. It is used for preliminary investigations in more limited areas and to supplement acoustic surveys. Also the refraction results need to be checked and calibrated by borings and/or sampling.

1.3.4 Soundings or simple borings (probings)

Soundings are performed by a rod with a special tip that is carried into the ground by rotation, hammering or by other mechanical means. The methods are termed according to the advancing method, such as rotary sounding, ram sounding, and motorized sounding or drilling by use of a range of equipment from hand-held engines to rock-drilling plant.

The recordings consist of the soil resistance against penetration:

(a) Rotary sounding: number of rotations/unit of penetration (for instance every 20 cm).
(b) Ram sounding: the number of blows/unit of penetration.
(c) Rotary/pressure sounding: continuous (automatic) registration of resistance at a specified penetration speed.
(d) Motorized sounding or drilling: the time spent/unit of penetration, and/or recorded resistance.

Different types of equipment are used worldwide. The penetration capacity depends on the strength of the rod, type and size of the tip, and the energy of the driving medium. Heavy rock-drilling equipment, as shown in Fig. 1.5, has very good penetration ability but needs a special flashing technique for deep penetration in soils. It is a suitable method for determining the boundary between soil and hard rock.

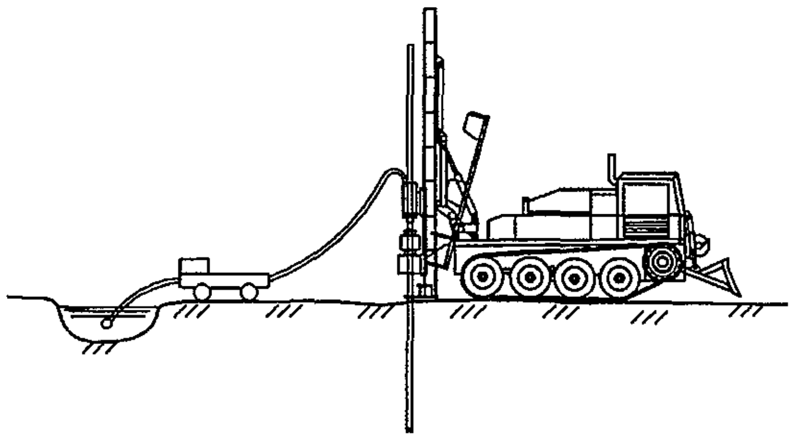

Fig. 1.5. Rock control drilling with water flashing

Sounding is a simple technique, which provides information of relative density, layering and thickness of soils. Sounding is suitable for preliminary investigations and needs to be supplemented by other investigations for calibration and for providing engineering data. Sounding is also a cheap method of obtaining additional data to assist interpolation between sampled boreholes.

1.3.5 Borings with in situ tests

In situ tests are used to measure certain characteristics or engineering properties of the ground. These methods avoid the disturbance or scale effect associated with testing of small samples in the laboratory.

Standard penetration test (SPT)

A standard penetration test (SPT) is historically the most common investigation method used worldwide, particularly in countries influenced by the USA. This test is undertaken in boreholes. A small diameter tube fitted with a cutting edge (split-spoon sampler) is driven into the soil below the base of a casing by means of a 65 kg hammer with a falling head of 700 mm. The sampling tube is driven 450 mm and the penetration resistance, or the N-value, is the number of blows required to drive the last 300 mm. During the driving, a disturbed sample is obtained which can be used for soil classification and testing of index properties.

Although there is no direct relation between the N-value and engineering properties, a range of empirical relations have been established, which are valid for different soil types and site conditions.

The SPT method has a good penetration capacity in hard layers. Other advantages are the availability of the equipment, and that it is well known and understood by construction contractors.

Cone penetration test (CPT)

A cone penetration test (CPT) provides a direct measurement of the resistance to the penetration of a thin rod with a conical tip. The resistance is measured in terms of both end bearing and side-friction components to the tip. The CPT penetrometer is advanced into the ground by a thrust machine placed at the surface or on a floating rig. There is also available CPT equipment designed to operate from the seabed, where the reaction force is provided by a heavy template, and normally used offshore at large water depths.

The resistance to penetration can be measured mechanically by pressure gauges at the surface at intervals of, say, 200 mm, or continuously by electric load cells mounted in the tip. Electric penetrometers can additionally be equipped to measure the poor water pressures induced in the soil as the tip advances and the reduction of excess pressure with time during a stop in the penetration.

The CPT method was invented in the Netherlands and is often called the Dutch cone penetration test. In recent years the method has also been developed in other countries. There are a lot of theoretical and empirical relationships available between the penetration results and soil types and engineering properties. Interpretation of the results requires experience.

The main advantage of this method is that it provides data that are otherwise only available through undisturbed sampling and laboratory testing, and normally at a lower cost. Disadvantages are a limited penetration capability in hard layers, and a limited availability and experience on the method in many regions.

Shear vane tests

The shear vane test is used to determine the undrained shear strength of cohesive soils. The vane consists of two blades fixed at right angles and attached to a rod, which is pushed into the soil from the surface, or from the bottom of a borehole. A calibration torque head is used to apply an increasing turning force to the rod until failure occurs. The measured strength may need to be corrected to allow for the effect of soil friction on the rods, anisotropy of the soil, certain scale effects and the soil plasticity. In particular, results of trial embankments show that in high plasticity clays the actual shear strength is less than the vane strength measured in situ, and empirical correction factors have been established.

1.3.6 Soil and rock sampling

Different boring methods are available to provide samples of the ground. The choice of method depends on the type of materials, the penetration depth required, and the requested size and quality of the samples. The methods can be divided into disturbed and undisturbed sampling.

Disturbed sampling

There are a great variety of methods and equipment types available worldwide and only the main types are mentioned below. The structure of the natural soil may be disturbed to a considerable degree by the action of the boring tool or excavation equipment, but the sample should be representative, i.e. keep a similarity to the material in situ. In addition to the SPT, mentioned under in situ tests, the following methods provide disturbed samples.

Test pits Excavation of pits by hand or machine is the simplest form of sampling, usually limited in depth to 3–6 m. Advantages are that the layering and behaviour of the soil can be observed in the field, and large samples can be taken for laboratory testing.

Auger borings Hand-operated or mechanical augers are cheap means of sampling in favourable soils without contents of stones.

Percussion methods Soil sampling can be performed by different methods of advancing a borehole into the ground, such as:

(a) Percussion, cable tool, or shell and auger borings, where the soil is loosened by various types of tools, and the soil is removed by shell or augers.
(b) Wash boring, where a stream of water or drilling mud removes the soil. To obtain samples, the soil in suspension needs to be settled in a pond or tank. This sampling procedure is unreliable

because certain soil fractions may be lost and the soil structure is lost.

The borehole stability is normally secured by a steel casing, which is pressed or driven down in steps as the hole is advanced. Uncased holes may be used if the walls are stable, such as in stiff clays, or if the walls are stabilized by a drilling mud. A tripod usually operates the casing and drilling tools.

Shallow offshore sampling may be done by gravity or vibro coring, which are rapid and cost-effective methods of investigating soft soils below the sea bottom:

(a) Gravity corers, penetrating by their own weight, and operated from a vessel.
(b) Vibro corers consisting of a frame carrying a sample tube, normally 75 or 100 mm diameter and up to 6 m long; operated by a crane from a support vessel. The sample tube is vibrated into the sea bottom and withdrawn while the sample is retained by a spring. Larger diameter and more sophisticated vibrocorers have been developed in recent years.

Rotary drilling Various types of rotary drilling are used to advance boreholes and to provide samples in hard soils and rock. Sampling is done by a tube (core barrel), fitted with diamond or tungsten cutting bits, which is rotated by a drilling rig and advances into the soil or rock by cutting an annulus of the material. A central core enters the barrel and is retained by a spring. The bit is cooled and lubricated by pumping water or a drilling fluid down the hollow drilling rods and into the bit. Core diameters are typically 50–80 mm while the length of the core barrels can vary from 1–3 m. Cores can be taken at intervals or continuously. They are normally placed in core boxes for inspection, description/classification and further testing.

Undisturbed sampling

Undisturbed samples represent, as closely as practicable, the true in situ structure and water content of the soil. The usual sampling method is to push or drive a thin-walled tube for its full length into the soil and then to withdraw the tube with its contents. The retracted tube is sealed at both ends and transported to the laboratory for opening and testing. Special equipment is developed to reduce the disturbance when pushing out the sample from the tube. There are basically two types of such samplers:

(a) Open tubes (Shelby tubes) that are operated from the bottom of uncased or cased boreholes. Particularly the upper part of the samples may be disturbed to some degree due to the effects of advancing the borehole.
(b) Various types of piston samplers, where a closed sampler is carried into the soil, and the tube pushed into the soil at the actual depth.

Length of the sample tubes is usually 50–80 cm. Undisturbed samples are typically taken at intervals of 1–2 m.

1.3.7 Field trials

Field trials can provide field data that are not obtained through boring methods, and are used to test foundation methods or field procedures prior to or integrated with the full-scale construction work.

Test piling and test loading Test piles can be driven in order to check the pile-driving resistance and depth penetration and/or the bearing capacity. Bearing capacity can be recorded by analysis of driving data through a device called the pile driving analyser (PDA). This procedure was developed in the early 1970s by a research team at the Case Western Reserve University, Ohio, USA ('Case method'). The data measured are the hammer impact force and acceleration during driving by use of strain transducers and accelerometers. The data are analysed in a computer program, originally known as the CAse Pile Wave Analysis Program (CAPWAP). The measured data include efficiency of the pile hammer, pile-bearing capacity along the pile and pile integrity. Any structural damage of the pile is indicated as well as the depth of such deficiency.

PDA equipment is nowadays available throughout the world and is frequently used during production piling as a routine by large piling contractors. PDA measurements have, to a large extent, replaced full-scale static test loadings because the PDA, besides being reliable, can be used on a great number of piles and is less costly.

Static pile test loading may be required under complex soil conditions, and is mandatory under certain codes such as those in the USA.

Trial dredging Data provided by ground investigation methods cannot be directly transformed to the performance of dredging plant. For large dredging undertakings, where geological conditions are complex or on the borderline of the capabilities of certain plant, trial dredging may therefore be advisable. Conditions triggering trial dredging include those where it has to be decided whether hard soil or rock can be dredged directly with available equipment or has to be pre-treated, for instance, by blasting. Trial dredging is expensive but it provides the best type of information if it is properly carried out and recorded.

Trial embankments Trial embankments are sometimes constructed in areas to be reclaimed in order to provide field data concerning the rate and magnitude of settlements and to test the effect of certain ground improvement methods. Trial embankments have historically been used in many cases to test vertical drains because the effect of such drains with respect to acceleration of settlements in complex soils cannot be calculated with certainty.

1.3.8 Laboratory tests

Laboratory tests on soil and rock samples can be divided into five groups as shown below:

(a) soil description and index (or routine) tests
(b) strength tests
(c) consolidation tests
(d) other physical tests
(e) mineralogical and chemical tests.

Index tests Index tests are generally simple and inexpensive tests, which provide basic information on the physical characteristics of the material, such as density, porosity, grain size, water content, plasticity, etc. The index properties are used in the soil classification and to provide engineering properties information through empirical relationships.

Description of the soil is often based on a standard classification method such as, for example, the AASHTO method or the Unified Soil Classification System. Such standard classification should be supplemented by a visual description of the soil. For dredging, PIANC has presented a Classification of Soils to be dredged, which is based on the British Standard. Classification of rock cores is normally done by visual description by an engineering geologist.

Strength tests Strength tests are used to determine the strength properties of soil and rock in compression, tension and shear. The most common test is the unconfined compression test, UCT, which can be used for clay and rock samples. Undrained strength in clay can also be tested by a falling cone or by a laboratory vane. More sophisticated testing is done in triaxial apparatus, either as drained or undrained triaxial tests, where the sample is subjected to a confining pressure. Various types of shear boxes are also used.

Consolidation tests Consolidation tests in oedometers are used to determine parameters for calculating magnitude and rate of settlements caused by embankment fills and structural loads.

Other physical tests Compaction tests are used to determine compaction properties and requirements of materials used in fills and pavements. Various material tests are performed to establish strength and durability of pavement materials, such as the Los Angeles Abrasion and Sodium Sulphate Soundness tests.

Mineralogical and chemical tests Rock samples are subjected to mineralogical and petrographical tests. Chemical tests are used to track contents of salts or other deleterious substances in concrete aggregates, for instance to indicate the risks of alkali–silica reactions.

1.4 Hydraulic laboratory studies

The use of hydraulic models should, in port planning and design of a port, generally be a standard part of all important port and harbour projects, where complex interactions between the berth structures within the mooring evaluation, and the waves and the seabed and coastline are involved. Although great strides have been, and will continue to be, made in mathematical and numerical modelling of such processes, laboratory models or field studies are still required to calibrate such methods, and physical models are often preferred in combination with mathematical models as a more expedient way of obtaining reliable answers.

Physical laboratory modelling and testing are commercial services that are provided by many commercial hydraulic laboratories, research institutions and universities around the world.

It is convenient to distinguish between the two-dimensional (2-D) and the three-dimensional (3-D) models.

The **3-D models** are models that cover a large area of sea bottom and shoreline, and where the width and breadth of the basin are of approximately the same magnitude, typically from 10–40 m. The point of

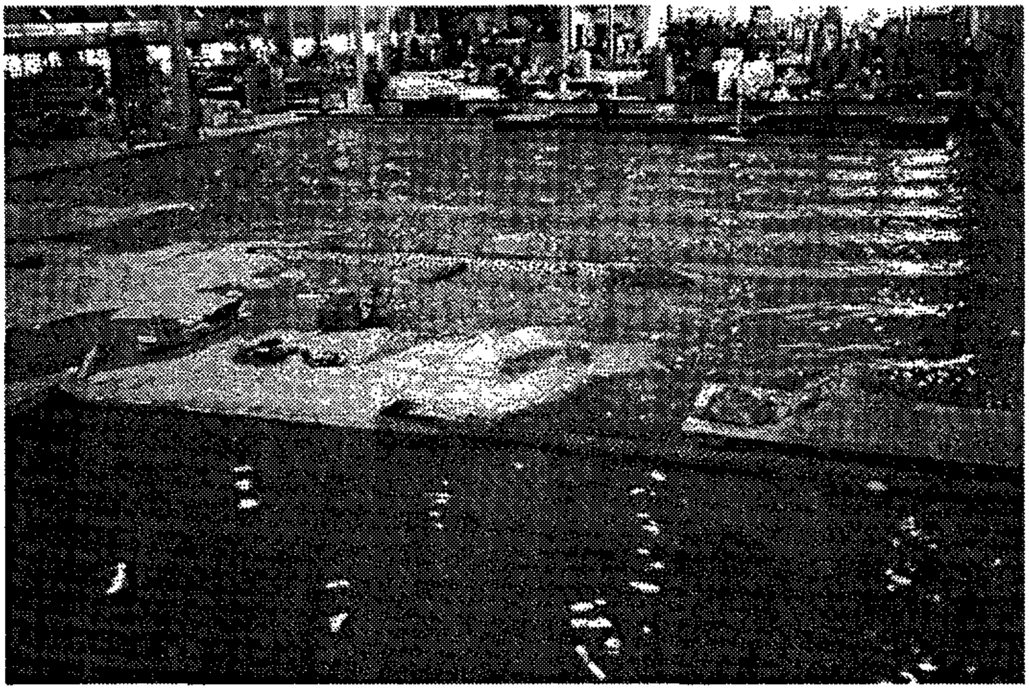

Fig. 1.6. A three-dimensional model of a fishing port. Photo courtesy of SINTEF, Norway

interest for a port, port entrance, breakwater or other type of structure, is built at one end of the basin and the wave generators are placed at the opposite end where near-deep water conditions prevail. Where an ocean or river current is required, this is usually introduced by letting water in at one end and draining it at the other.

Figure 1.6 shows a three-dimensional model at a scale of 1:100 of a fishing port at Sirevaag in south Norway. The water depth in the model is 0.55 m. The laboratory model was used to determine wave height distributions along the proposed new breakwater (background), and to determine wave heights and ship motions at berth inside the new harbour basin. The wave generators at the far end of the model basin produce wave spectra that have been calculated using a numerical refraction model.

Such tests may be used for a number of tasks, such as:

(a) determining the wave height distribution along a breakwater
(b) assessing the effects on wave agitation of different types of structures or civil works in a port, such as breakwaters, dredging, piers or artificial beaches
(c) assessing the quality of planned berths by measuring wave heights and (preferably) ship motions at berth
(d) locating wave-breaking zones.

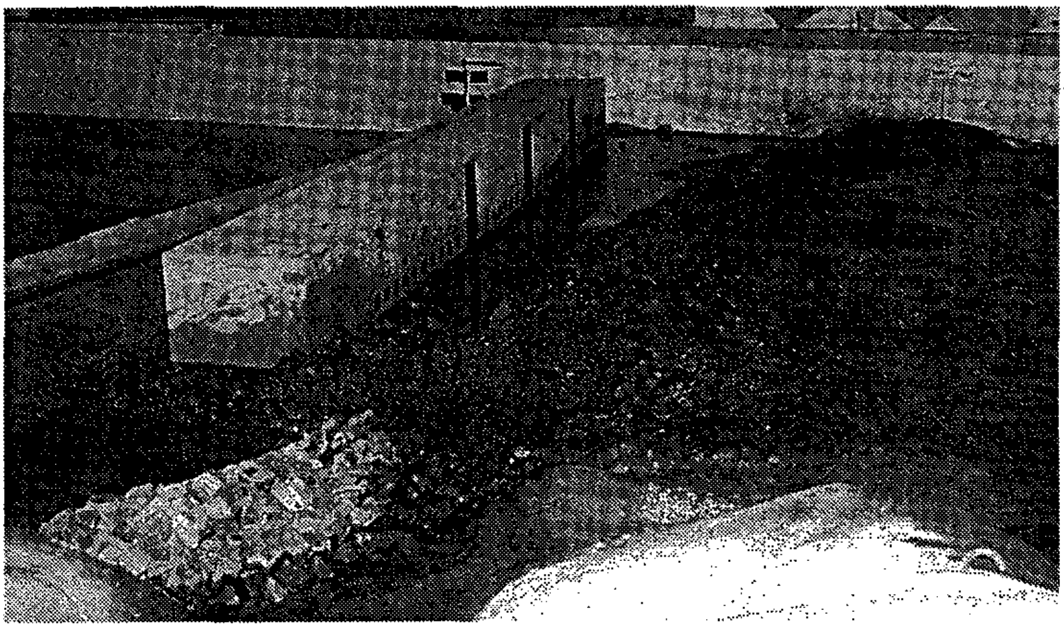

Fig. 1.7. A two-dimensional model of a breakwater. Photo courtesy of SINTEF, Norway

The **2-D models** are models where one horizontal dimension is minimized so that the basin has the shape of a canal or a flume. When applying waves in a 2-D model, it is assumed that the direction has been established by other means, so that wave height, length and breaking are the only parameters. General wave flumes may be up to several hundred meters in length, but more typical facilities for port engineering purposes are 30–100 m long and 1–10 m wide, with depths of 1–4 m.

Figure 1.7 shows an almost 2-D model of wave attack on a breakwater and industrial area at Gismeroy, Norway. The breakwater was modelled at a scale of 1:50 in a wave canal which was 4.5 m × 60 m × 1.5 m.

Examples of tasks that are resolved in wave flumes using a 2-D model are:

(a) stability testing of rubble-mound breakwaters and concrete-block breakwaters
(b) wave run-up and overtopping of breakwaters
(c) wave forces on structures such as breakwaters, pilings, pipelines, etc.

Scale and model effects are the results of factors that are either not reproduced correctly in the model, or due to factors that have been introduced in the model that are not present in nature. A typical model effect is the introduction of basin walls. The walls will create reflected wave that are not present in nature, and action must be taken to either minimize them through passive or active absorbers, or account for their effect when analysing the final results. Scale effects include, for example, viscosity and surface tension. While these effects can be assumed to be negligible when considering wave attack on a several ton armour unit in a breakwater, these effects grow more important as the model scale decreases. Since there is no practical alternative to using plain water in the model tests, there are no ways of eliminating these effects. The preferred method of reducing and controlling the scale effects is to keep the model scale as large as the infrastructure in the laboratory permits.

Wave generators are used to create waves. A number of types and methods exist, but the simplest and most commonly used principle is a flat paddle, which is moved horizontally to create the waves. Monochromatic wave generators (i.e. capable of only one wave period at a time) are obsolete and are not widely used.

Modern facilities have random wave generators that are hydraulically, pneumatically or electrically powered. Depending on the type, the wave generator may also be fitted with software and sensors that automatically compensate for reflected waves, so that the operator may specify a given wave height, which will then be produced from the generator. If no such compensation exists, a calibration procedure must be followed.

Three-dimensional wave generators create waves that closely replicate real sea states by combining waves of different heights, periods and directions in one sea state. Such waves are, however, of limited use in port design studies because the field in which the whole array of waves exists is very limited. These wave generators are commonly used to study wave impact in very small areas, such as wave forces on a slender structure as shown in Fig. 1.8.

An example of a 3-D model used to investigate wave loading on a breakwater, wave agitation in the approach channel and wave agitation inside a port is as shown in Fig. 1.8. The ship model was used for illustration purposes only, and was not part of the testing programme. The scale was 1 : 100, and the water depth at a distance of 2 km from the port is in the range 8–10 m (8–10 cm in the model). Thus, in order to generate proper waves, the entire model was built on a platform which lets the waves form and be generated in a 50 cm model water depth. The stability of the breakwater was tested in a separate 1 : 50 model.

The guidelines on commonly used values for the most requested port-related studies are shown in Table 1.1.

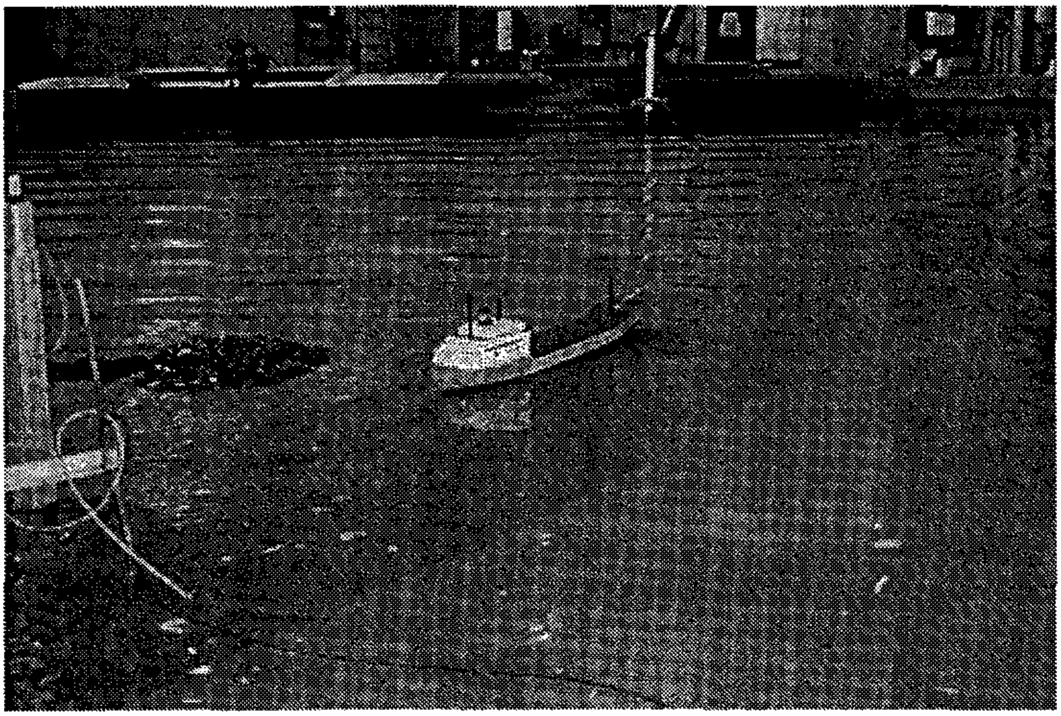

Fig. 1.8. A three-dimensional model for investigating wave loading. Photo courtesy of SINTEF, Norway

Table 1.1. Guidelines on commonly used values

Type of study	Type of model	Scale range	Primary parameters	Other
Wave loading on breakwaters, distribution	3-D	1:80–1:120	Wave height, period and direction, water depths, water level	Waves must be allowed to generate and form in near-deep-water conditions
Wave agitation inside ports	3-D	1:80–1:100	Wave height, period and direction, water depths, water level	
Breakwater stability, behaviour of individual stones in a breakwater	2-D, 3-D if space permits	1:20–1:60	Wave height and period, breaker type, water level	
Wave forces on structures, seawalls, piles, etc.	2-D	1:10–1:40	Wave height and period, breaker type, water level	

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2

Environmental forces

2.1 General

Consideration should be given to the environmental forces and conditions at all stages of a berth structure, both during the construction phase and during the service life. One should bear in mind the variable and often unpredictable character of all environmental forces that can act on a berth structure; it would be unrealistic to expect substantial cost savings by attempting to design the structure for a shorter design life.

Generally it is not easy to forecast what the maximum environmental forces on a berth or ship will be. It is dependent on the direction of the wind and waves, the current, the size and type of ship, tugboat assistance, and whether the ship is loading or unloading, etc.

It is always recommended that when dealing with environmental forces on important berth structures or forces on larger ships that a hydraulic institute with experiences in coastal engineering, for example, be consulted.

When evaluating the environmental forces it is necessary to obtain estimates of the expected extreme conditions that may act on the port site area. These estimates can be obtained by observations and calculations of high wind speeds, and are then applied to a forecasting technique for later use in a mathematical or physical model.

2.2 Wind

In Fig. 2.1 an example of a **windrose** diagram showing yearly distribution of wind directions in decadegrees and the forces in percentage of the time. The dominating or prevailing wind directions are, in this case, southeasterly and northwesterly. In Fig. 2.2 the same wind forces are

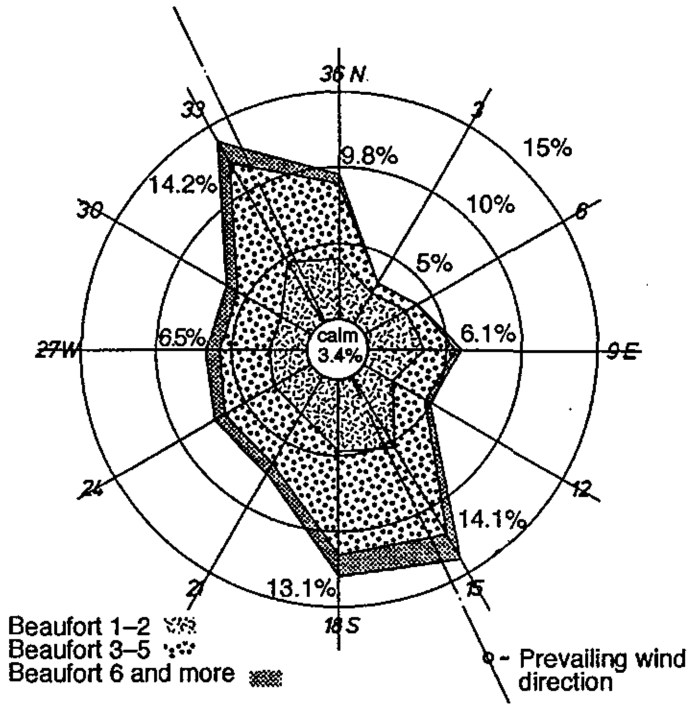


Fig. 2.1. An example of a windrose for the yearly distribution of the wind direction and forces in percentage of time

shown as the frequency of the yearly wind forces for each Beaufort interval.

The wind forces are classified in accordance with the Beaufort wind scale. The **Beaufort** range of intensity from 0–12 is shown in Table 2.1.

The mean wind velocity and direction should, in accordance with the Beaufort scale, be recorded 10 m above mean sea level and should be based upon the 10-min averages of the wind velocity and direction.

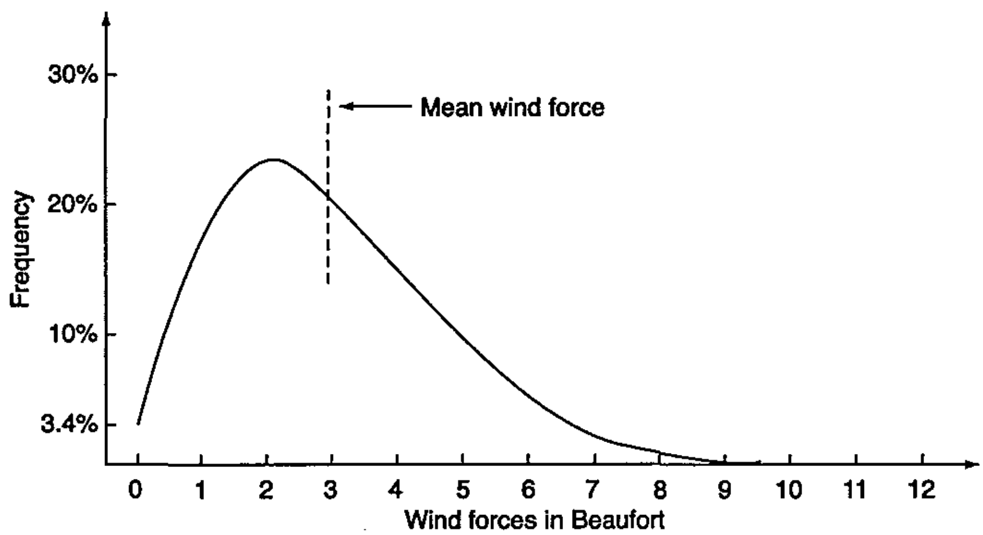


Fig. 2.2. The frequency of yearly wind forces

Table 2.1. The Beaufort wind scale

Beaufort	Description	Velocity	
		m/sec	knots
0	Calm	0.0–0.2	0–1
1	Light air	0.3–1.5	1–3
2	Light breeze	1.6–3.3	4–6
3	Gentle breeze	3.4–5.4	7–10
4	Moderate breeze	5.5–7.9	11–16
5	Fresh breeze	8.0–10.7	17–21
6	Strong breeze	10.8–13.8	22–27
7	Near gale	13.9–17.1	28–33
8	Gale	17.2–20.7	34–40
9	Strong gale	20.8–24.4	41–47
10	Storm	24.5–28.4	48–55
11	Violent storm	28.5–32.6	56–63
12	Hurricane	32.7–	64–

The Norwegian Petroleum Directorate has recommended that the maximum wind speed, averaged over short time periods, may be obtained by multiplying the actual 10-min mean wind speed by the gust factors as shown in Table 2.2.

Provided that no specified information on the gust ratio is available, i.e. the ratio between the short-period wind speeds to the mean wind speed for the wind conditions at the site, PIANC recommend that Table 2.2 could tentatively be applied for larger wind velocities. Table 2.3 shows the relationship between the 1-h mean wind speed and the associated maximum speeds for a range of shorter mean duration:

The gust factor will depend upon the topographical conditions around the harbour basin and the port location. If there is a lack of proper wind

Table 2.2. Relationship between 10 min mean wind speed and gust factor

Wind duration	Gust factor
3 s mean	1.35
10 s mean	1.30
15 s mean	1.27
30 s mean	1.21
1 min mean	1.15
10 min mean	1.0

Table 2.3. Relationship between one hour mean wind speed and gust factor

Wind duration	Gust factor
3 s mean	1.56
10 s mean	1.48
1 min mean	1.28
10 min mean	1.12
30 min mean	1.05
1 h mean	1.00

information in the area, use of the gust factors in Table 2.3 is recommended.

For port and ship operation it is claimed that gust durations shorter than about 1 min will be of secondary importance, but at some oil terminals around the North Sea it has been observed that wind durations down to 20–30 s may affect tankers in a ballasted condition.

2.2.1 Wind forces

The wind forces acting on a ship may vary considerably, as do the current forces, with both the types and sizes of ships, and should therefore best be established by testing in a hydraulic institute. This is especially so for the wind forces acting upon a ship with a large windage sided area, e.g. fully loaded container ships or large passenger ships which are influenced greatly, while very large oil tankers have large variations in the longitudinal forces depending upon the shape or design of the bow. Generally the wind effects on port and harbour operations are more important than those of the wave and current.

In this chapter some different methods and national standards and regulations' methods for calculations of the wind forces are compared against each other. It should be noted that in important evaluations these standards and regulations should only be used as a guide to the magnitude of the forces on the ship.

The magnitude of the wind velocity V_w to be applied in design varies from place to place and has to be assessed in each case. The design wind velocity should correspond to the maximum velocity of the gusts that will affect the ship, and not only to the average velocity over a period of time. A 30-sec average wind velocity is recommended for use in the wind force equations for mooring analyses.

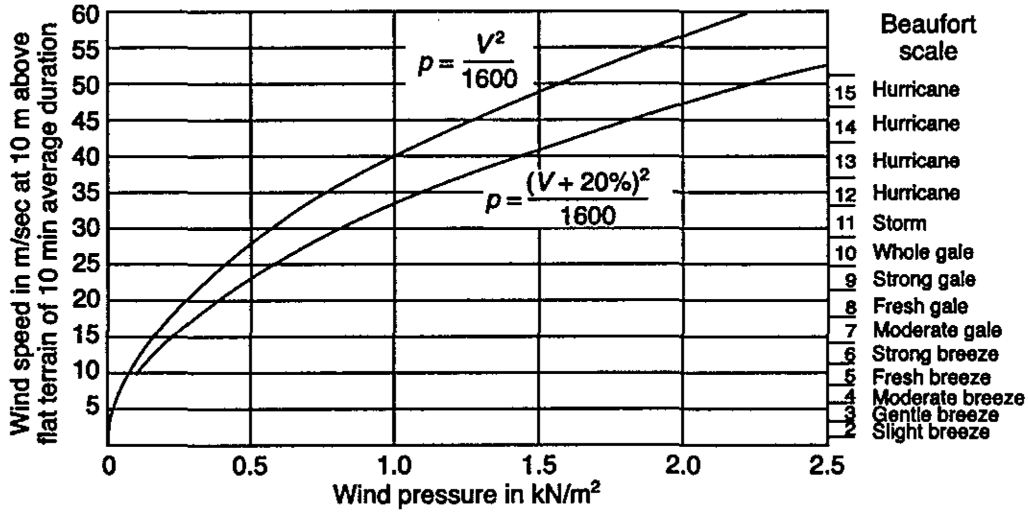


Fig. 2.3. Wind pressure and wind velocity

These gust velocities can be about 20 per cent higher than the average velocity. In the case of moored ships, the gust duration must be sufficient for the full mooring line or fender strains to develop, taking into account the inertia of the ship. This can lead to a reduced design wind speed. It should also be taken into account that the wind area is not symmetrical to the midship line, which implies the development of a moment of rotation.

Figure 2.3 shows the relationship between wind pressure and wind velocity over a 10-min period. When reading the wind pressure, the curve with gust factor, $V + 20$ per cent should be selected due to the wind gust factor.

Lower wind velocity than 30 m/s, after the Beaufort scale, with a gust factor of 1.2 should not be assumed for design of berth structures, i.e. minimum wind pressure should be 0.81 kN/m^2. If the wind velocity increases above about 25–30 m/s the ship would normally either leave the berth or take in ballast to reduce its wind area.

It is very important to note that the wind force is proportional to the square of the wind velocity.

2.2.2 Different wind standards and recommendations

Below, some different methods and national standards and regulations' methods for calculating the wind forces are shown and compared against each other. It should be noted that these standards and regulations should only be used as a guide to the magnitude of the forces on the ship. If more accurate wind force calculations are needed these should be established by model testing in a hydraulic institute.

A **general standard** formula for calculation of the wind forces on a ship moored at a berth structure:

$$P_w = C_w \times (A_w \times \sin^2\varphi) + (B_w \times \cos^2\varphi) \times \gamma_w \times \frac{V_w^2}{2g}$$

$$P_w = C_w \times (A_w \times \sin^2\varphi) + (B_w \times \cos^2\varphi) \times \frac{V_w^2}{1600}$$

$$P_w = C_w \times (A_w \times \sin^2\varphi) + (B_w \times \cos^2\varphi) \times p$$

where

P_w = wind force in kN
C_w = wind force coefficient
A_w = laterally projected area of ship above water in m^2
B_w = front area of ships above water in m^2
φ = angle of wind direction to ship's centreline
γ_w = specific gravity of air 0.01225 kN/m^3 at 20 °C
V_w = velocity of the wind in m/s. It is recommended to use a 30-s average wind velocity
g = acceleration of gravity 9.81 m/s^2
p = is the wind pressure in kN/m^2.

The maximum wind force in the above equation is when $\varphi = 90°$, i.e. the wind blow perpendicular to the ship's centreline.

$$P_w = C_w \times A_w \times p$$

The magnitude of C_w depends on the shape of the ship above water and the orientation of the ship related to the wind direction. As average values of C_w for isolated ships, the following are recommended: for wind crosswise to the ship $C_w = 1.3$, for wind dead against the bow $C_w = 0.9$ and for wind dead against the stern $C_w = 0.8$.

The **Spanish Standard ROM 0.2-90** recommends that the wind forces or pressure on a ship will be:

The resultant wind force in kN:

$$R_v = \frac{\rho}{2g} \times C_V \times V_W^2 \times ((A_t \times \cos^2\alpha) + A_l \sin^2\alpha)$$

$$= C_V \times ((A_t \times \cos^2\alpha) + A_l \sin^2\alpha) \times \left(\frac{V_w^2}{1600}\right)$$

$$tg\varphi = \frac{A_l}{A_t} \times tg\alpha$$

The lateral or transverse wind force in kN:

$$F_T = R_V \times \sin\varphi$$

The longitudinal wind force in kN:

$$F_L = R_V \cos\varphi$$

where

A_l = broadside or lateral projected wind area in m^2
A_t = head on or transverse projected wind area in m^2
ρ = specific weight of air 1.225×10^{-3} t/m^3
α = angle between the longitudinal axes of the ship from bow to stern, and the wind direction against the ship
φ = angle between the ship's longitudinal axes from bow to stern, and the resultant wind force R_v on the ship
C_v = shape factor for the ship varies between 1.0 and 1.3. Lacking of no precise determination from, for example, model studies the value 1.3 shall be used
V_w = the design wind velocity in m/s at a height of 10 m with the shortest interval (gust) that will overcome the ship's inertia shall be adopted as the basic wind velocity. A mean velocity corresponding to the following gust values should be adopted:
- 1 min for ship length larger than or equal to 25 m
- 15 s for ship length less than 25 m.

The **British Standard BS 6349: Part 1: 2000**, recommends that the wind forces or pressure on a ship will be:

The lateral or **transverse** wind force in kN:

$$F_{Twind} = (C_{TWforward} + C_{TWaft}) \times \rho \times A_L \times \frac{V_W^2}{10\,000}$$

The **longitudinal** wind force in kN:

$$F_{Lwind} = C_{LW} \times \rho \times A_L \times \frac{V_W^2}{10\,000}$$

where

A_L = broadside or lateral or transverse projected wind area in m^2
C_{TW} = transverse wind force coefficient forward or aft depending on the angle of wind as shown for ballasted condition in Table 2.4 and for loaded condition in Table 2.6
C_{LW} = longitudinal wind force coefficient depending on the angle of wind as shown for ballasted condition in Table 2.4 and for loaded condition in Table 2.6

ρ = specific weight of air varying from 1.3096 kg/m^3 at 0 °C to 1.1703 kg/m^3 at 30 °C
V_W = design wind speed in m/s at a height of 10 m above water level. It is recommended for design of the moorings to use 1-min mean wind speed.

The **OCIMF: Oil Companies International Marine Forum 1997** recommend that the wind force or pressure on an oil tanker will be:

The **lateral** wind forces:

$$F_{Ywind} = F_{YAwind} + F_{YFwind}$$

The lateral wind force at aft perpendicular:

$$F_{YAwind} = C_{YAw} \times \rho_w \times A_L \times \frac{V_w^2}{2012}$$

The lateral wind force forward perpendicular:

$$F_{YFwind} = C_{YFw} \times \rho_w \times A_L \times \frac{V_w^2}{2012}$$

The **longitudinal** wind force:

$$F_{Xwind} = C_{Xw} \times \rho_w \times A_T \times \frac{V_w^2}{2012}$$

where

A_L = broadside or lateral projected wind area in m^2
A_T = head-on projected wind area in m^2
C_{YAw} = the transverse wind force coefficient aft depending on the angle of wind as shown for ballasted condition in Table 2.4 and for loaded condition in Table 2.6
C_{YFw} = the transverse wind force coefficient forward depending on the angle of wind as shown for ballasted condition in Table 2.4 and for loaded condition in Table 2.6
C_{Xw} = the head-on wind force coefficient depending on the angle of wind as shown for ballasted condition in Table 2.4 and for loaded condition in Table 2.6
ρ_w = specific weight of air 1.223 kg/m^3 at 20 °C
V_W = the design wind speed in m/s at a height of 10 m above water level. It is recommended to use a 30-sec average wind velocity
2012 = conversion factor for velocity in knot to m/sec.

The **OCIMF/SIGTTO:** *Oil Companies International Marine Forum/ Society of International Gas Tanker & Terminal Operators Ltd 1995*, recommend the wind force or pressure on a gas tanker will be:

The **lateral** wind forces:

$$F_{Yw} = F_{YAw} + F_{YFw}$$

The lateral wind force at aft perpendicular:

$$F_{YAw} = C_{YAw} \times \rho_w \times A_L \times \frac{V_w^2}{2012}$$

The lateral wind force forward perpendicular:

$$F_{YFw} = C_{YFw} \times \rho_w \times A_L \times \frac{V_w^2}{2012}$$

The **longitudinal** wind force:

$$F_{Xw} = C_{Xw} \times \rho_w \times A_T \times \frac{V_w^2}{2012}$$

where

A_L = broadside or lateral or transverse projected wind area in m^2
A_T = head-on projected wind area in m^2
C_{YAw} = the transverse wind force coefficient aft depending on the angle of wind as shown in Table 2.4 and for loaded condition in Table 2.6
C_{YFw} = the transverse wind force coefficient forward depending on the angle of wind as shown for ballasted condition in Table 2.4 and for loaded condition in Table 2.6. The coefficient will be the same both in ballasted and loaded conditions since the differences in the gas tankers' loaded conditions are not significant due to the relative small change in draft from ballasted to fully loaded conditions
C_{Xw} = the head-on wind force coefficient depending on the angle of wind as shown for ballasted conditions in Table 2.4 and for loaded condition in Table 2.6. The coefficient will be the same in both ballasted and loaded conditions since the differences in the gas tankers' loaded conditions are not significant due to the relative small change in draft from ballasted to fully loaded conditions
ρ_w = specific weight of air 1.248 kg/m^3 at 20 °C

V_W = the design wind speed in m/sec at a height of 10 m above water level. It is recommended to use a 30-sec average wind velocity

2012 = conversion factor for velocity in knot to m/s.

Comparison between the different wind standards and recommendations for ships in ballasted condition

For a comparison between the different wind standards and recommendations, the different approximate wind force coefficients on ships in ballast condition are shown in Table 2.4.

Example of wind forces on ships in ballasted conditions

Based on the different wind standards and recommendations' formulas and the different wind force coefficients in Table 2.4 with a gust factor recommended by the different standards and recommendations, examples for comparison of the different wind forces in kN with the wind acting longitudinal (0° against the bow) and lateral (90° to the ship longitudinal axis) for different wind speeds in m/s for ships in ballasted condition and with a 95 per cent confidence limit, after Chapter 20, are shown in Table 2.5 with the following ship dimensions:

(a) Oil tanker 200 000 dwt
(b) $L_{OA} = 350$ m, $L_{BP} = 346$ m, $B = 56.2$ m
(c) Broadside or lateral ballasted projected wind area $A_L = 6930\,\text{m}^2$
(d) Head-on or transverse ballasted projected wind area $A_T = 1730\,\text{m}^2$

For a comparison between OCIMF and OCIMF/SIGTTO, the OCIMF/SIGTTO coefficients for a membrane tanker have been used.

As may be seen from the calculations, the wind forces vary considerably between the different standards and recommendations. The above methods for calculation should therefore only be used as a guide to the magnitude of the forces. Since the wind forces can vary considerably both in type and size of ship, one should therefore, to obtain the most accurate forces for important berth structures, wind test on scale models. This is especially important for wind forces upon high-sided ships.

Comparison between the different wind standards and recommendations for ship in loaded condition

For a comparison between the different wind standards and recommendations, the different approximate wind force coefficients on

Table 2.4. Wind coefficient in **ballasted condition**

Wind coefficients on tankers and container ships in ballast condition

Angle from bow to stern	British Standard BS 6349 Part 1: 2000						OCIMF 1997		
	Very large tankers			Large container ships without deck load			Oil tankers		
	C_{TWf}	C_{TWa}	C_{LW}	C_{TWf}	C_{TWa}	C_{LW}	C_{YFw}	C_{YAw}	C_{Xw}
Forw. 0°	0.0	0.0	1.2	0.0	0.0	0.7	0.00	0.00	−0.88
15°	0.7	0.3	0.9	0.8	0.4	0.7	0.16	0.06	−0.68
30°	1.4	0.9	0.7	1.8	1.0	0.7	0.30	0.16	−0.48
45°	2.1	1.4	0.4	2.5	1.7	0.5	0.42	0.30	−0.25
60°	2.4	1.9	0.2	2.9	2.2	0.3	0.50	0.38	−0.12
75°	2.4	2.3	0.1	3.1	2.6	0.4	0.48	0.46	−0.05
90°	2.2	2.6	0.0	2.9	2.9	0.4	0.43	0.52	0.00
105°	1.8	2.8	−0.2	2.7	3.0	0.3	0.35	0.55	0.14
120°	1.3	2.7	−0.4	2.2	3.1	0.1	0.25	0.54	0.30
135°	0.8	2.5	−0.6	1.7	2.9	−0.2	0.15	0.48	0.42
150°	0.4	1.8	−0.7	1.0	2.0	−0.5	0.07	0.36	0.55
165°	0.1	1.0	−0.8	0.4	1.2	−0.6	0.03	0.20	0.62
Aft 180°	0.0	0.0	−0.8	0.0	0.2	−0.6	0.00	0.00	0.62

Wind coefficients on tankers and container ships in ballast condition

Angle from bow to stern	OCIMF/SIGTTO 1995					
	LNG membrane tankers			LNG spherical tankers		
	C_{YFw}	C_{YAw}	C_{Xw}	C_{YFw}	C_{YAw}	C_{Xw}
Forw. 0°	0.00	0.00	−1.02	0.00	0.00	−1.02
15°	0.12	0.07	−0.95	0.12	0.07	−0.95
30°	0.29	0.20	−0.80	0.30	0.19	−0.80
45°	0.44	0.35	−0.56	0.51	0.33	−0.56
60°	0.52	0.45	−0.30	0.61	0.45	−0.30
75°	0.52	0.52	−0.10	0.60	0.55	−0.10
90°	0.46	0.59	0.02	0.55	0.62	0.02
105°	0.38	0.64	0.15	0.46	0.64	0.15
120°	0.30	0.65	0.39	0.35	0.62	0.48
135°	0.20	0.60	0.65	0.22	0.50	0.83
150°	0.10	0.45	0.79	0.12	0.34	0.97
165°	0.04	0.20	0.89	0.04	0.15	1.02
Aft 180°	0.00	0.00	0.90	0.00	0.00	1.00

Table 2.5. Comparison of wind forces on ship in ballasted condition with the different standards

Forces due to wind with gust factor in ballast condition										
Wind speed in m/sec	General formula Forces in kN		Spanish Standard ROM 0.2-90 Forces in kN		British Standard BS 6349 Part 1: 2000 Forces in kN		OCIMF 1997 Oil tankers Forces in kN		OCIMF/SIGTTO LNG membrane 1995 Forces in kN	
	0°	90°	0°	90°	0°	90°	0°	90°	0°	90°
10	140	810	186	745	144	576	133	576	158	650
15	315	1824	418	1675	324	1296	300	1297	355	1462
20	560	3243	744	2979	567	2304	533	2305	630	2600

ships in loaded condition are shown in Table 2.6. The wind force coefficients for OCIMF/SIGTTO are applicable to draft conditions ranging from ballasted to fully loaded.

Example of wind forces on ships in loaded conditions

Based on the different wind standards and recommendations' formulas and the different wind force coefficients in Table 2.6 with a gust factor recommended by the different standards and recommendations, examples for comparison of the different wind forces in kN with the wind acting longitudinal (0° against the bow) and lateral (90° to the ship's longitudinal axis) for different wind speeds in m/sec for ships in a loaded condition and with a 95 per cent confidence limit, after Chapter 20, are shown in Table 2.7 with the following ship dimensions:

(a) Oil tanker 200 000 dwt
(b) $L_{OA} = 350\,\text{m}$, $L_{BP} = 346\,\text{m}$, $B = 56.2\,\text{m}$
(c) Broadside or lateral fully loaded projected wind area $A_L = 4300\,\text{m}^2$
(d) Head on or transverse fully loaded projected wind area $A_T = 1210\,\text{m}^2$

For a comparison between OCIMF and OCIMF/SIGTTO, the OCIMF/SIGTTO coefficients for a membrane tanker have been used.

As may be seen from the examples above, there are significant differences in the wind forces between the various wind standards and

Table 2.6. Wind coefficients in **loaded condition**

Wind coefficients on tankers and container ships in loaded condition

Angle from bow to stern	British Standard BS 6349 Part 1: 2000						OCIMF 1997		
	Very large tankers			Large container ships with deck load			Oil tankers		
	C_{TWf}	C_{TWa}	C_{LW}	C_{TWf}	C_{TWa}	C_{LW}	C_{YFw}	C_{YAw}	C_{Xw}
Forw. 0°	0.0	0.0	1.8	0.0	0.0	0.6	0.0	0.0	−0.98
15°	0.2	0.3	1.7	0.6	0.4	0.6	0.04	0.06	−0.88
30°	0.5	0.8	1.4	1.6	0.9	0.6	0.1	0.16	−0.72
45°	0.9	1.4	1.1	2.4	1.6	0.5	0.2	0.32	−0.5
60°	1.1	1.7	0.7	2.8	2.1	0.3	0.24	0.38	−0.34
75°	1.2	2.0	0.3	2.8	2.5	0.4	0.25	0.42	−0.15
90°	1.1	2.1	0.0	2.6	2.7	0.4	0.24	0.46	0.04
105°	1.0	2.2	−0.3	2.3	2.8	0.3	0.21	0.49	0.19
120°	0.8	2.3	−0.6	1.9	2.8	−0.1	0.17	0.49	0.3
135°	0.6	2.1	−0.8	1.5	2.6	−0.5	0.12	0.47	0.4
150°	0.4	1.7	−1.2	0.9	1.8	−0.8	0.07	0.36	0.62
165°	0.2	1.0	−1.4	0.3	0.8	−0.5	0.04	0.2	0.75
Aft 180°	0.0	0.0	−1.4	0.0	0.0	−0.5	0.0	0.0	0.75

Wind coefficients on tankers and container ships in loaded condition

Angle from bow to stern	OCIMF/SIGTTO 1995					
	LNG membrane tankers			LNG spherical tankers		
	C_{YFw}	C_{YAw}	C_{Xw}	C_{YFw}	C_{YAw}	C_{Xw}
Forw. 0°	0.00	0.00	−1.02	0.00	0.00	−1.02
15°	0.12	0.07	−0.95	0.12	0.07	−0.95
30°	0.29	0.20	−0.80	0.30	0.19	−0.80
45°	0.44	0.35	−0.56	0.51	0.33	−0.56
60°	0.52	0.45	−0.30	0.61	0.45	−0.30
75°	0.52	0.52	−0.10	0.60	0.55	−0.10
90°	0.46	0.59	0.02	0.55	0.62	0.02
105°	0.38	0.64	0.15	0.46	0.64	0.15
120°	0.30	0.65	0.39	0.35	0.62	0.48
135°	0.20	0.60	0.65	0.22	0.50	0.83
150°	0.10	0.45	0.79	0.12	0.34	0.97
165°	0.04	0.20	0.89	0.04	0.15	1.02
Aft 180°	0.00	0.00	0.90	0.00	0.00	1.00

Table 2.7. Comparison of wind forces on ship in loaded condition with the different standards

Forces due to wind with gust factor in loaded condition										
Wind speed in m/sec	General formula		Spanish Standard ROM 0.2-90		British Standard BS 6349 Part 1: 2000		OCIMF 1997 Oil tankers		OCIMF/ SIGTTO LNG membrane 1995	
	Forces in kN		Forces in kN		Forces in kN		Forces in kN		Forces in kN	
	0°	90°	0°	90°	0°	90°	0°	90°	0°	90°
10	98	503	130	462	89	357	93	358	110	403
15	221	1132	293	1040	201	804	210	805	248	907
20	392	2012	520	1848	357	1430	373	1430	441	1613

recommendations. Therefore in the design of, for example, mooring structures one must be very careful which standard to use.

2.2.3 Wind area and wind loads

The area of ship above water, part projected on a plane perpendicular to the wind direction, varies greatly, not only due to the different sizes of ships, but even more to the different types of ships, and also depending on whether the ship is being loaded or not. As an example, a modern general cargo ship of 30 000 ton displacement, fully loaded, has a wind area of about 10 m^2/lin m of ship, while the same ship in ballast condition has an area of about 14 m^2/lin m of ship. Large passenger ships will have wind areas of about 26 m^2/lin m or more.

How high the design wind velocity should be assumed to be will depend on the location of the berth structure, but it is generally not justified to use a higher value than 40 m/s with gust factor when calculating the mooring forces. Without closer investigations, i.e. Beaufort number 13, which corresponds to a pressure of about 1.5 kN/m^2. Therefore assuming too small a wind velocity could be critical, keeping in mind that in the wind loading formula the velocity occurs to the second power. Table 2.8 gives a guideline for wind loads for design purposes.

For **piers** where ships can berth on both sides, the total wind load acting on the pier should be the wind load on the largest ship plus 50 per cent of the wind load on the ship on the other side of the pier.

Table 2.8. Wind loads for design purposes

Ship displacement in tons up to	Wind load in kN/m of ship
2000	10
5000	10
10000	15
20000	20
30000	20
50000	25
100000	30

If the wind blows at an angle to the ship, there will be a transverse and a longitudinal load component plus a possible moment on the berth structure. The pressure on a ship of length L_S caused by wind and/or current must be transmitted to the berth structure over the berth length L_Q. The load or pressure on the berth structure should therefore equal the pressure on the ship's side $\times L_S/L_Q$.

It should be noted that if a ship is protruding outside the end of a berth structure, the contact length, L_Q, could be very small. The wind and/or the current will also try to turn the ship around the corner of the berth structure.

2.3 Waves

Waves are traditionally, and for practical reasons, classified into the following different types of waves:

(a) **Wind waves** or **locally generated waves**. These are generated by winds that are acting on the sea surface bordering on the port site.
(b) **Swell** or **ocean waves**. These are normally also wind generated waves, but are created in the deep ocean at some distance from the port site, and the wind that created them may be too distant to be felt in the port or may have stopped blowing or changed its direction by the time the waves reach the port.
(c) **Seiching** or **long waves**. Waves of this type have very long periods — typically from 30 s up to the tidal period 12 h 24 min — and are mostly found in enclosed or semi-enclosed basins, such as artificial port basins, bays or fjords.
(d) **Waves from passing ships**. Ship waves may be a significant problem in certain ports, especially since they are generated by a

moving source and may appear in areas where large waves would not be expected. Ship waves may also be very complex.

(e) **Tsunamis** and waves created by large, sudden impacts, such as earthquakes, volcanoes or landslides that end up in the ocean.

Waves are classified into the following:

(a) **Deep-water waves** are waves in which the ratio water depth d/wave length L is ≥ 0.5
(b) **Intermediate-water waves** are waves in which d/L is <0.5 and >0.04
(c) **Shallow-water waves** are waves in which d/L is ≤ 0.04
(d) **Breaking waves** are waves, which, for example, fall forward since the forward velocity of the crest particles exceeds the velocity of the propagation of the wave itself. In deep water this normally occurs when $L < 7\,H$, and in shallow water when the water depth d is approximately equal to $1.25\,H$. The still-water depth, where the wave breaking commences, is called the breaking depth.

Wind generated waves are defined by their height, length and period. The height, length and period are dependent on the fetch (the distance the wind blows over the sea in generating the waves), and the velocity, duration and direction of the wind. The wave characteristics for deep-water waves are shown in Fig. 2.4. The wave period is the time between successive crests passing a given point. The wave steepness is defined as

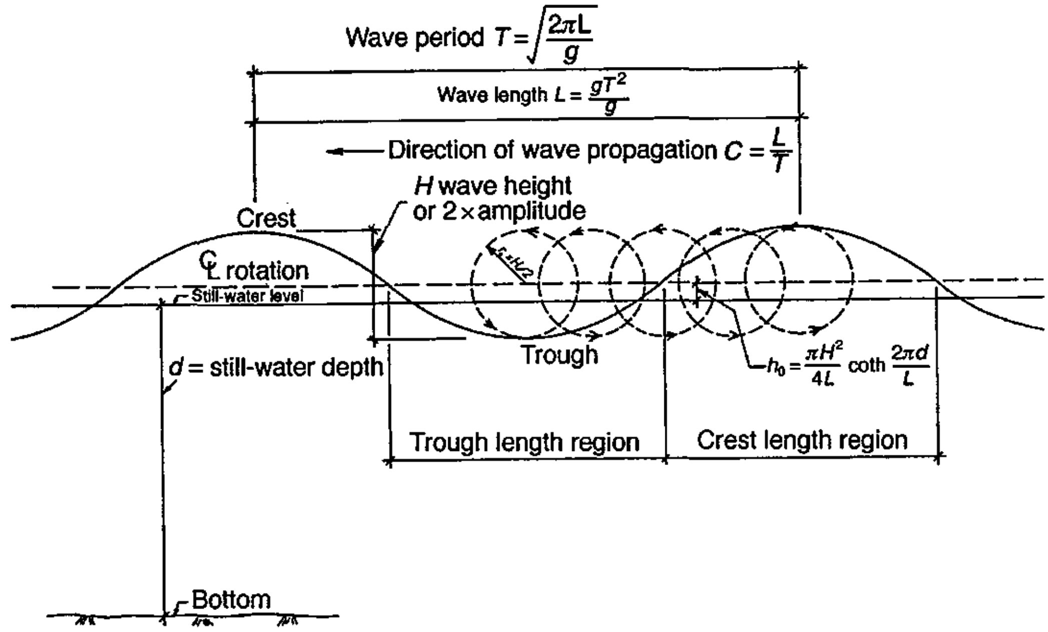


Fig. 2.4. Wave characteristics in deep water

the wave height divided by its length. As the waves propagate in water, it is only the waveform and part of the energy of the waves that move forward.

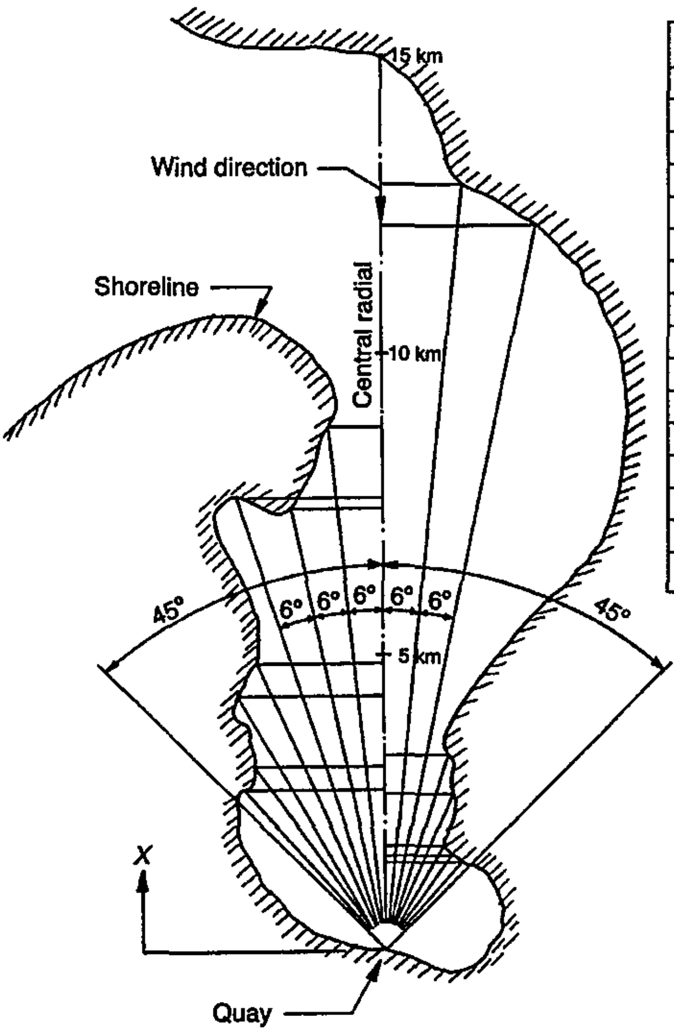
The **wave heights** H may be defined as follows:

(a) H_m is the arithmetical mean value of all recorded wave heights during a period of observation, $0.6H_s$
(b) H_s is the significant wave height; it is the arithmetical mean value of the highest one-third of the waves for a stated interval
(c) $H_{1/n}$ is the average value of the $1/n$ highest waves in a series of waves, usually of length 15–20 min. Commonly used values of n are 3 (significant wave height), 10, 100
(d) $H_{1/10}$ is the arithmetical mean value of the height of the highest 10 per cent $= 1.27H_s$
(e) $H_{1/100}$ is the arithmetical mean value of the height of the highest 1 per cent $= 1.67H_s$
(f) $H_{\max}$ is the maximum wave height $= 1.87H_s$ or rounded to $= 2H_s$ when a high risk of danger is present, or if storms of long duration are to be considered

The variables in wind wave height computations are:

V_{10} = the wind speed at 10 m above sea level, usually taken as a 10 min mean which is representative of the entire fetch and the entire duration of the situation
F = the fetch length
t = the duration of the wind.

Accurate calculations of the wave heights at the end of a **fetch** require a detailed knowledge of the fetch and the wind field. A procedure for determining the effective fetch distance is shown as an example in Fig. 2.5. It consists of constructing 15 radials from the berth site at intervals of 6°, limited by an angle of 45° on either side of the wind direction. These radials are extended from the berth site until they first intersect the shoreline, as shown in the figure. The length component of each radial in the direction parallel to the wind direction is measured and multiplied by the cosine of the angle. The resulting values for each radial are added together and divided by the sum of the cosines of all the individual angles. Where the fetch region is rectangular with a relatively uniform width, Fig. 2.6 may be used to obtain the effective fetch length. As shown from these examples, fetches limited by landform will be significantly lower than fetches over more open waters, and will result in lower wave generation.



α	$\cos\alpha$	X	$X\cos\alpha$
42°	.743	2.7	2.0
36°	.809	2.9	2.3
30°	.866	4.2	3.6
24°	.914	4.8	4.4
18°	.951	7.6	7.3
12°	.978	7.4	7.2
6°	.995	9.8	9.8
0°	1.000	15.0	15.0
6°	.995	12.7	12.6
12°	.978	12.0	11.7
18°	.951	3.3	3.2
24°	.914	2.6	2.4
30°	.866	1.7	1.5
36°	.809	1.5	1.3
42°	.743	1.5	1.1
Total 13.512			85.4

$$F_{\text{eff}} = \frac{\Sigma X\cos\alpha}{\Sigma\cos\alpha}$$

$$\frac{85.4}{13.512} = 6.3\text{ km}$$

Fig. 2.5. Calculation of effective fetch for irregular shoreline

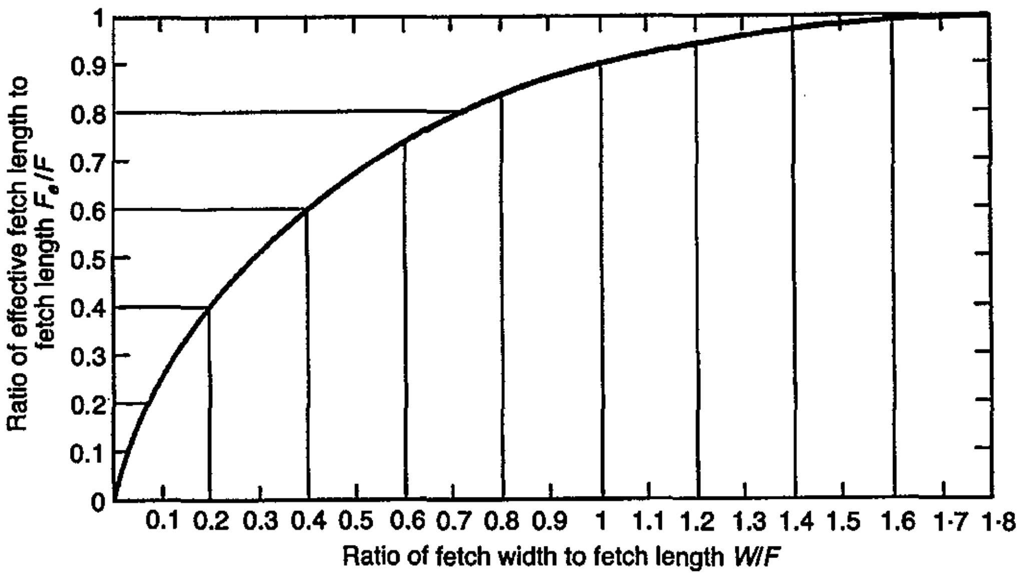


Fig. 2.6. Effective fetch length for rectangular fetch region

Generally the waves are generated in the transfer of energy from the air moving over the sea surface by the following ways:

(a) The first variations in the sea or water level are created due to the reactions of the water level to the small pressure difference in the moving air. These will increase by the pressure difference exerted by the moving wind on the front and on the back of the wave.
(b) Tangential stress between the water and the air which are moving at different speeds relative to each other.

It is clear that the wave characteristics will be a function of the wind velocity, since both the pressure difference and the tangential stress are functions of the wind velocity. Waves generated by wind of a certain velocity will have only attained the characteristics typical for this wind velocity after the wind has blown for a certain time. Generally the waves will at first increase rapidly in size and then grow at a decreasing rate the longer the wind lasts.

In Fig. 2.7 the significant wave height, period and the necessary minimum wind duration for constant wind velocity are shown as a function of the fetch for deep-water conditions in more practical curves.

In Fig. 2.8 the significant wave height is shown both as a function of wind duration for constant wind velocity and unlimited and limited fetch for deep water in more practical curves.

In the figures the significant wave height is in m, wave period in s, the wind speed in m/sec, the fetch in m and the wind duration in s or h. For practical wave predictions it is usually satisfactory to regard the wind speed as reasonably constant if the variations do not exceed 2.5 m/s (5 knots) from the mean.

The water depth will have an effect on the wave generation. For a given wind velocity and fetch conditions, the wave heights will be smaller and the wave periods shorter if the generation takes place in intermediate and shallow-water depth rather than in deep-water depth. In Fig. 2.9 the significant wave height is shown as a function of fetch and wind velocity with unlimited wind duration for intermediate and shallow-water depth, and with an assumed bottom friction factor equal to 0.01.

As an example of the use of the curves, the following cases are shown below:

(a) Case A: from Fig. 2.7 with an effective fetch length of 20 km and the constant wind velocity of 20 m/s, the significant wave height is 1.90 m, the significant period is 5.35 s and the necessary wind

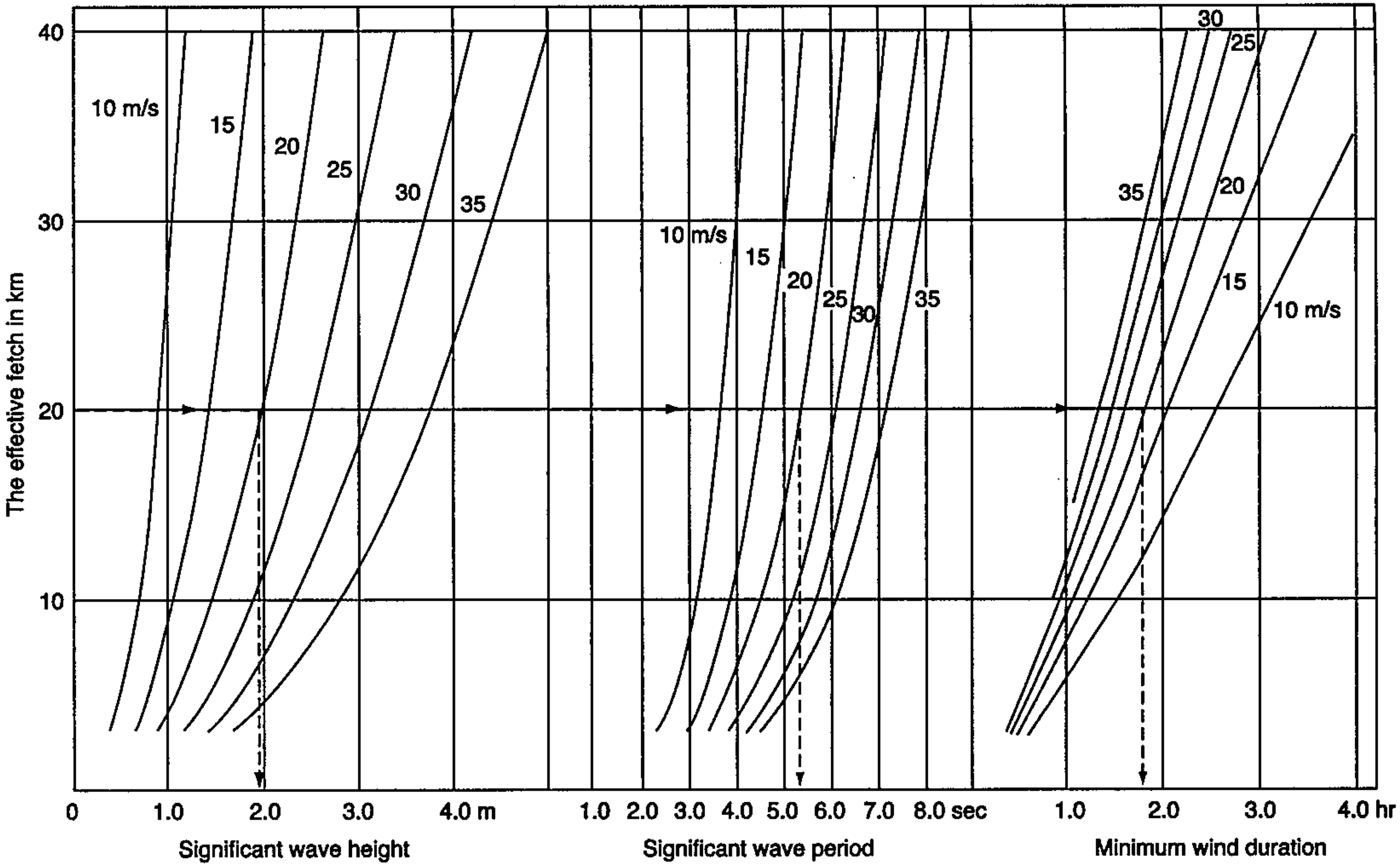


Fig. 2.7. Significant wave height, period and duration for constant wind velocity as a function of fetch for deep water condition

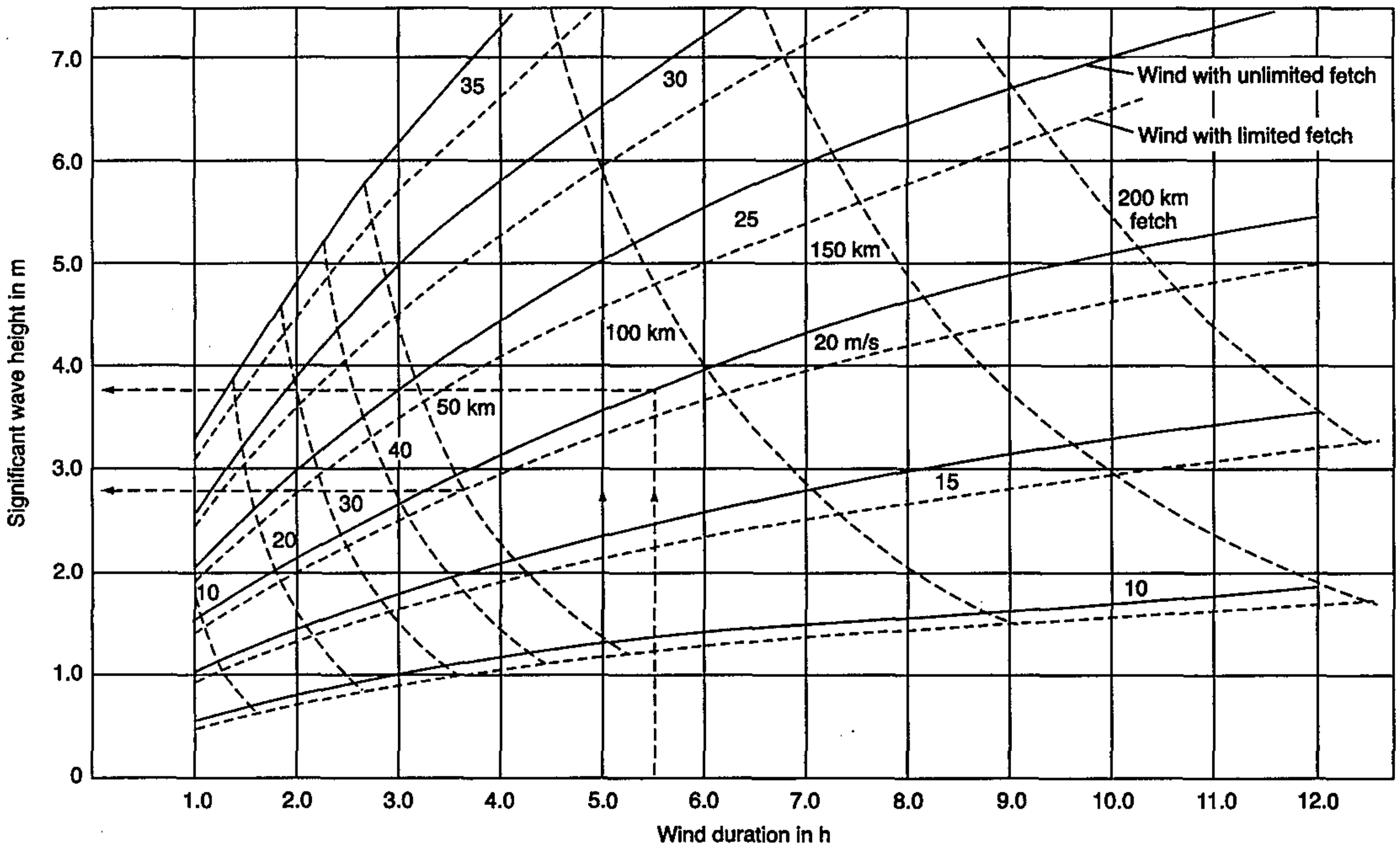


Fig. 2.8. Significant wave height as function of wind duration for constant wind velocity and for unlimited and limited fetch for deep-water condition

condition

$$\frac{gH_s}{V^2} = 0.283 \tanh\left[0.530\left(\frac{gd}{V^2}\right)^{0.75}\right] \tanh\left[\frac{0.0125(gF/V^2)^{0.42}}{\tanh\left[0.530(gd/V^2)^{0.75}\right]}\right]$$

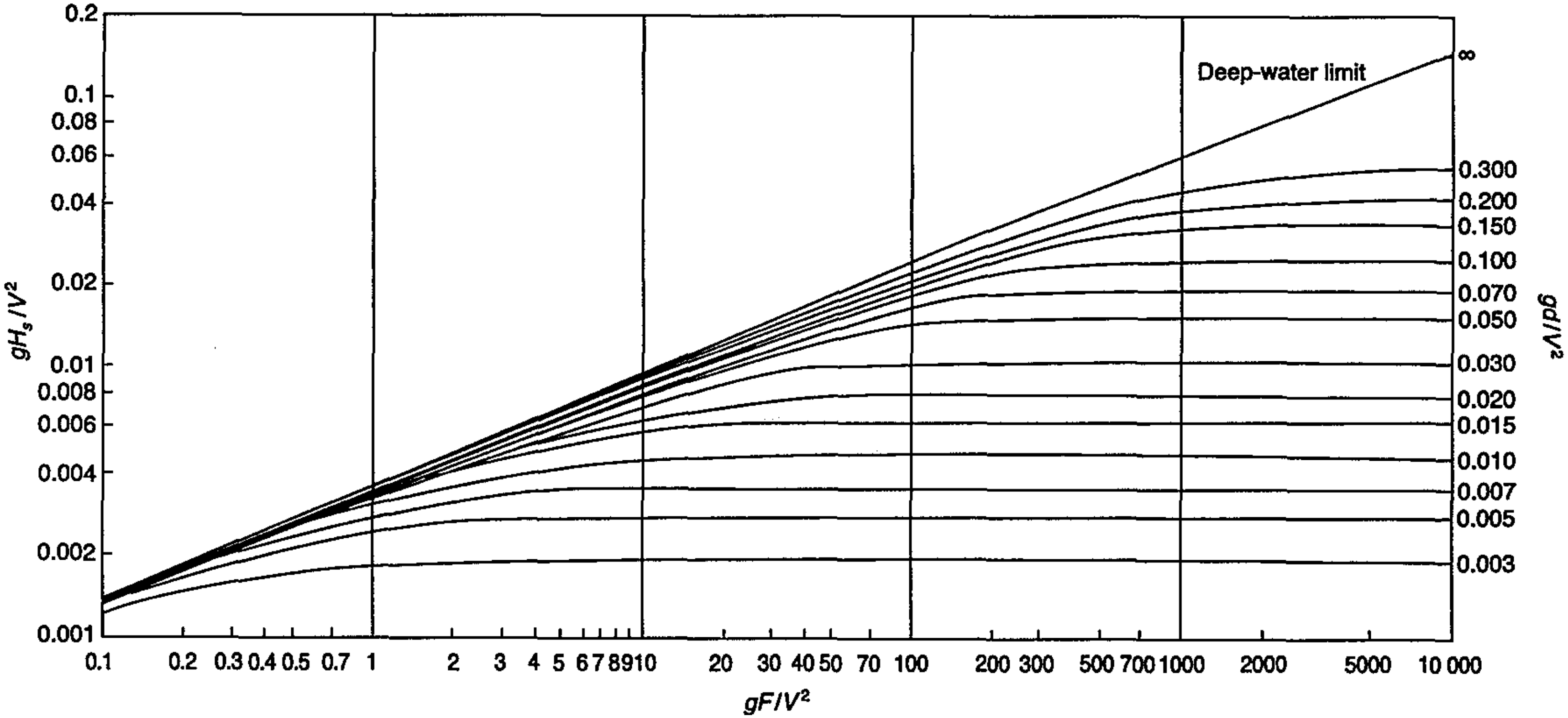


Fig. 2.9. Significant wave height H_S as a function of fetch F and wind velocity V with unlimited wind duration for intermediate and shallow water of constant depth

duration to develop the significant wave height for that particular fetch length is 1.80 h.

(b) Case B: from Fig. 2.8 with wind duration of 5.5 h for a constant wind velocity of 20 m/s in an unlimited fetch area, the significant wave height is found to be 3.80 m.

(c) Case C: from Fig. 2.8 with wind duration of 5.0 h for a constant wind velocity of 20 m/s with a limited fetch of 50 km, the significant wave height is found to be 2.80 m.

(d) Case D: the effective fetch for an irregular shoreline as shown in Fig. 2.5 is found to be 6.3 km. With a wind velocity of 25 m/s, and unlimited wind duration the significant wave height for deep water is found to be 1.60 m. If the average constant water depth along the central radial is 10 m and the bottom friction factor is assumed to be 0.01, the significant wave height from Fig. 2.9 is found to be $H_s = 0.022 \times V^2/g = 1.40$ m.

2.3.1 Waves near ports

Where it is necessary to carry out instrumental wave recording, it is advisable to install the recording system as early as possible to enable the recording programme to be as long as possible. A minimum recording time should be one year to enable reasonably reliable data because any shorter duration is unlikely to yield a representative set.

In most cases, the waves that constitute the design wave condition in a port are a combination of ocean waves and locally generated wind waves. Where the port is situated in sheltered waters such as a bay or in a fjord, the distinction between the two types of waves and the reason for treating them separately is quite obvious. On open coastlines, however, the two types may become inseparable, and one may choose to consider ocean waves only.

A diagram showing a suggested method for calculating wave height is shown in Fig. 2.10.

Local wind waves and ocean waves are traditionally calculated separately and added by adding the energy components of the two sea states:

$$H_{s,i} = (H_{s,w}^2 + H_{s,o}^2)^{1/2}$$

where subscript i denote combined inshore waves, w wind waves and o ocean waves.

If one of the wave types is totally dominant over the other, one may choose to ignore the contribution from the lesser component.

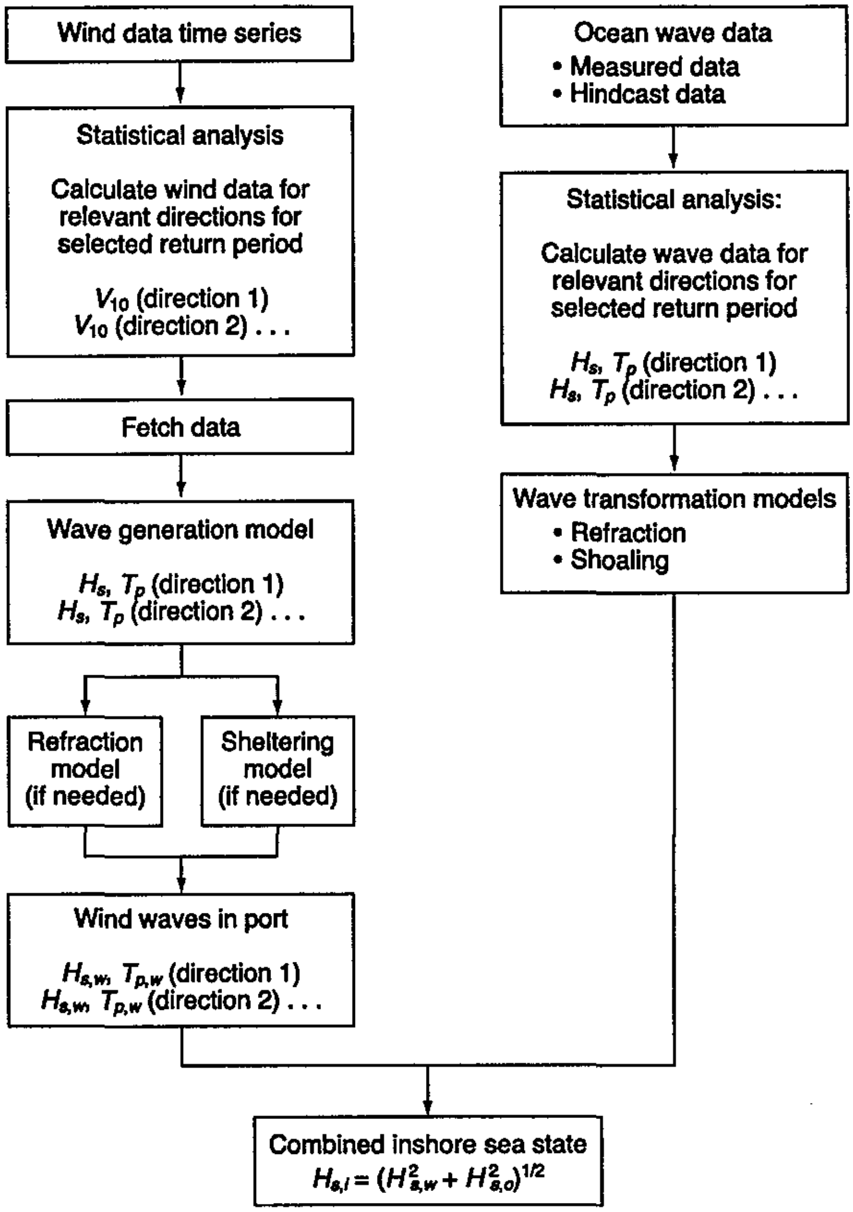


Fig. 2.10. Procedure for calculating wave heights

2.3.2 Breaking waves

Wave breaking occurs when the wave crest travels faster than the rest of the waveform and becomes separated from it. From the port designer's point of view, wave breaking induced by limited water depth is the most relevant type of breaking.

A first approximation of the breaker height can be obtained from the simple expression:

$$d_b = 1.28H_b$$

where d_b is the depth at breaking, H_b is the individual wave height at breaking.

2.3.3 Wave action

Since most berth structures are sheltered against sizable waves, the statical calculations of berth structures normally do not explicitly deal with forces and reactions due to wave action. Such forces are assumed to be taken care of by the very fact that the structure is also designed for impact and mooring forces. For breakwaters, and similar structures heavily exposed to waves, the wave actions must of course be studied very closely in each case.

The characteristics of the waves in a port or berth area and their effect on the berthing structure are influenced by the following factors:

(a) Bathymetry (bottom topography) in the vicinity of the berth structure itself.
(b) Waves can be reflected from the near-shore slopes.
(c) Refraction of waves can or will take place as the waves enter shallow water at an inclined angle.
(d) Wave shoaling will influence the design wave heights as the waves enter shallow water.
(e) The wave regime in the harbour area will be due to swell and or wind generated waves.

Structures, which are very resistant to overload stresses, may be designed for a wave height that is lower than H_{max}. This increase in risk for, for example, flooding or destruction can be justified by the lower cost of construction as long as it does not decrease the safety of personnel. Therefore, one should always evaluate both the construction costs and the capitalized maintenance costs. If it does not decrease the safety of personnel it can, in some cases, be cheaper, e.g., to apply a shore protection which requires regular maintenance work instead of constructing an expensive and maintenance-free shore protection.

Figure 2.11 and Fig. 2.12 show wave actions along a breakwater where the top of the breakwater is too low.

2.3.4 Design wave

The design wave may be chosen by:

Selecting a design wave **return period** (R_p). In this context, the design return period is defined as the average time lapse between two consecutive events where the wave height is equal to or greater than a given significant wave height (the design wave height). This method is often chosen for its simplicity, and is preferred for smaller structures

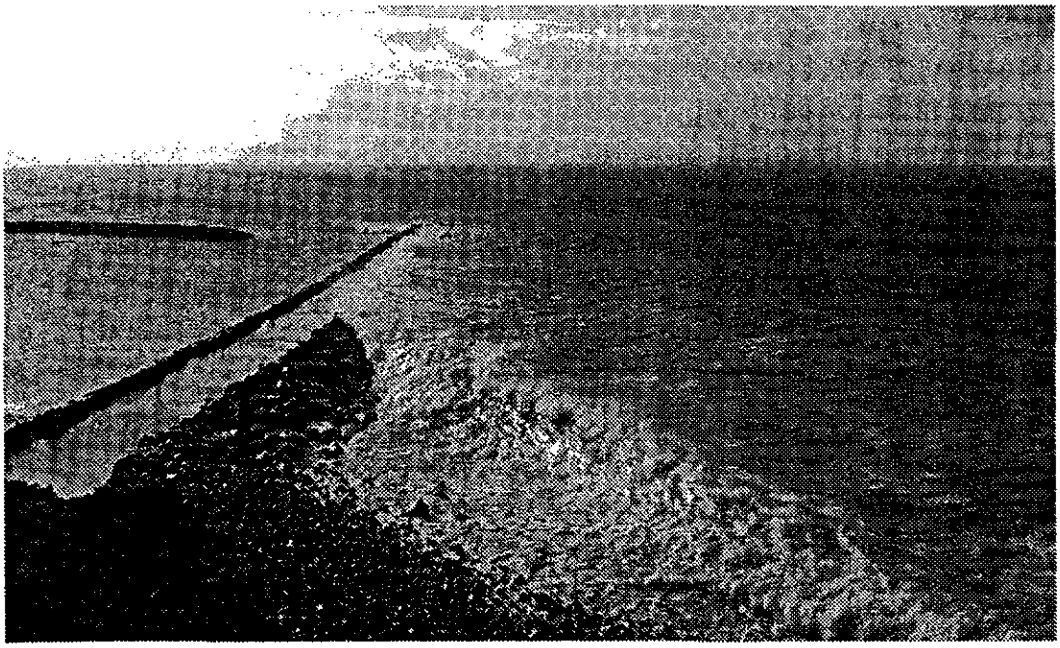


Fig. 2.11. Wave action along a breakwater

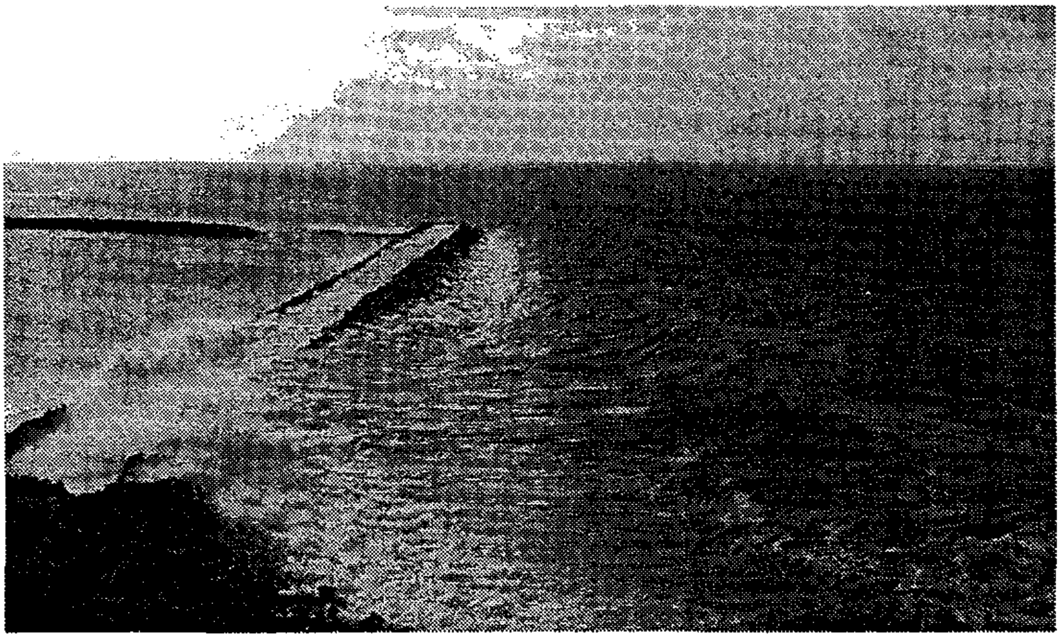


Fig. 2.12. Wave action along a breakwater

such as breakwaters, piers, small ports, etc. The design return period should be in the range 50–100 years for simple technical structures.

When selecting the return period, attention must be paid to the consequences of encountering an exceedance of the design wave height. Low return periods (in some cases as low as 25 years) are chosen when the consequences of an exceedance are minor or easily repaired. Where the economical consequences are great, or human life and health may be threatened, the normal return period is a minimum of 100 years. One may also specify a differentiated set of design return periods for different types of functions and types of structures in a port.

Specifying a required technical **lifetime** of the structure; it should be noted that the design life is not necessarily the same as the return period of the design. If a lifetime or design working life N of a structure is specified, then it is assumed that N number of years must pass from the start of the project without the structure experiencing a situation (i.e. significant wave height) that exceeds its design load. As such the designer cannot give a guarantee, one is therefore forced to introduce a probability P of encountering such a situation over the projected lifetime.

The relationship is given as:

$$T = \frac{1}{1 - \sqrt[n]{1 - \dfrac{P}{100}}}$$

where

T = the design return period
n = the design lifetime
P = the probability (in %) of encountering a significant wave height greater than the design wave height during the lifetime N.

The relationship between the design working life, return period and probability of wave heights exceeding the normal average is shown in Fig. 2.13.

A port structure is required to have a design lifetime of 50 years. If the associated probability of exceedance of the design wave height level is set equal to 10 per cent, then the required design return period is approximately 500 years. By setting the design return period equal to the design life, e.g. $T = n = 50$ years, Fig. 2.13 shows that there is a 63 per cent chance of exceeding the design wave height over the next 50 years. Likewise, by setting the design return period equal to twice the lifetime (100 years), the probability of exceedance during the lifetime is approximately 40 per cent. Therefore it is necessary to evaluate the consequences and probabilities of damage against the costs of reducing or avoiding these risks.

The return period of the wave should be at least 100 years but not less than the design life of the structure. If the consequences from failure of the structure are so grave as to be unacceptable at other than very low probabilities, the structure should be able to withstand design conditions with return periods of 1000 years or more.

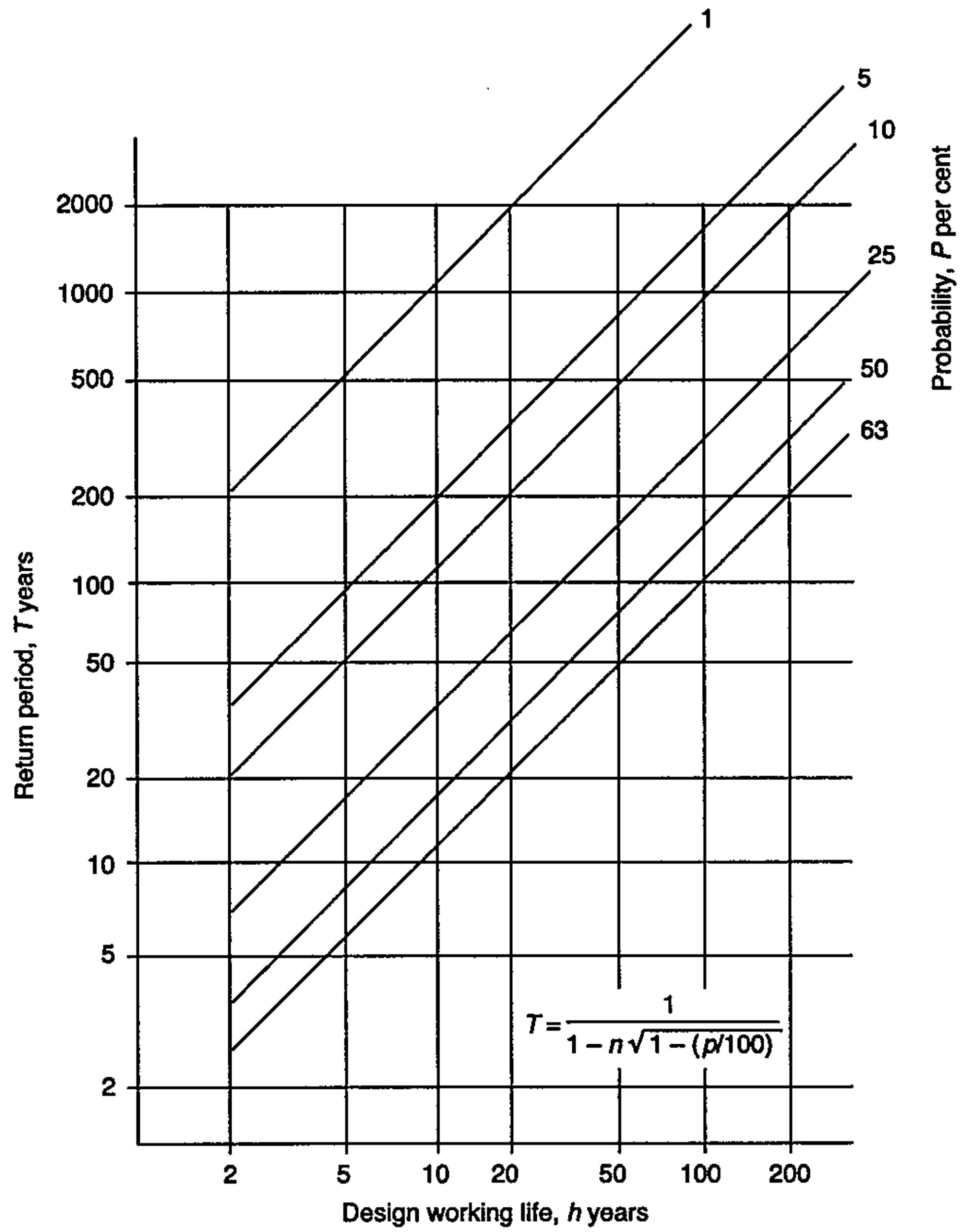


Fig. 2.13. The relationship between design working life, return period and probability of wave

The **design wave height** H_{des}, which should be chosen for the design, may, depending on the severity of the allowable risk, be as in Table 2.9.

2.3.5 Wave forces

For wave forces which can act on a ship the **Spanish Standard ROM 0.2-90**, recommends that the wave forces or pressure on a ship will be:

Table 2.9. Design wave height

Type of structure	H_{des}/H_s
Erosion protection	1.0–1.4
Rubble-mound breakwater	1.0–1.5
Concrete breakwater	1.6–1.8
Berth structures	1.8–2.0
Structure with high safety requirements	2.0

The lateral or **transverse** wave force in kN:

$$F_{Twave} = C_{fw} \times C_{dw} \times \gamma_w \times H_s^2 \times D' \times \sin^2 \alpha \times 10$$

The **longitudinal** wave force in kN:

$$F_{Lwave} = C_{fw} \times C_{dw} \times \gamma_w \times H_s^2 \times D' \times \cos \alpha \times 10$$

where

$$D' = L_{bp} \times \sin \alpha + B \times \cos \alpha$$

where

C_{fw} = waterplane coefficient depending on the wave longitude L_w at the location and the ship's draught D. If $(2\pi/L_w) \times D$ is more than 1.4, $C_{fw} = 0.064$, and if $(2\pi/L_w) \times D$ is less than 0.2, C_{fw} is 0.0
C_{dw} = depth coefficient depending on the wave longitude L_w and the water depth at the location. If $(4\pi/L_w) \times h$ is more than 6.0, $C_{dw} = 1.0$, and if $(4\pi/L_w) \times h$ is equal to 0.0, C_{dw} is 2.0
γ_w = specific gravity of water — seawater 1.034 t/m^3 — fresh water 1.00 t/m^3
H_s = the design significant wave height
α = the angle between the longitudinal axis of the ship, considered from bow to stern and the direction of the wave
D' = the projection of the ship length in the direction of the incident waves
L_{bp} = the length between perpendiculars
B = the beam of the ship
D = the draught of the ship
h = the water depth at the location.

2.4 Current

The magnitude and direction of the tidal current and the wind-generated current must be evaluated to establish any influence on the berthing and unberthing operations.

Current can arise in a port basin due to wind transporting water masses, differences in temperature and salt contents, tidal effects, water flow from river estuaries, etc. At some Norwegian harbours, situated at the mouth of a river, the currents are known to have reached a velocity of 3 m/s or 6 knots.

When designing new berth structures it is always important to ensure that the berth front is directed as parallel as possible to the prevailing current. Since the direction of the current can vary, it is also necessary to investigate over a longer time period the magnitude of the current perpendicular to the direction of the berth front. Should such a component reach a value of about 0.5 m/s perpendicular to, for example, an open pier, the berthing operation would be very difficult.

Even if currents do not usually set up loads of vital importance to, for example, a finished berth structure, they can still be of importance during the construction of the berth. For instance, the normal driving of piles is hardly possible if the current velocity is higher than 1.5 m/s. Divers will hardly be able to work properly if the current velocity is higher than 0.5 m/s.

Tidal current must be measured at various depths in the harbour area. The tidal current is referred to as a flood current on a rising tide and an ebb current on a falling tide.

The magnitude of **wind-generated current** will in the open sea be approximately 1–2 per cent of the wind speed at 10 m above the water level.

Berthing structure and the mooring equipment for oil and gas tankers should generally be at least capable of resisting loads due to any one of the following current conditions acting simultaneously with the design wind from any of the following directions: 1.5 m/s or 3 knots at 0° and 180° (current parallel to the berth), 1.0 m/s or 2 knots at 10° and 170°, and for pier structure 0.4 m/s or 0.75 knots from direction of maximum beam current loading.

2.4.1 Current forces

The current forces acting on a ship may vary considerably with both types and sizes of the ship, and should therefore best be established by testing. The longitudinal current forces are especially very scale dependent.

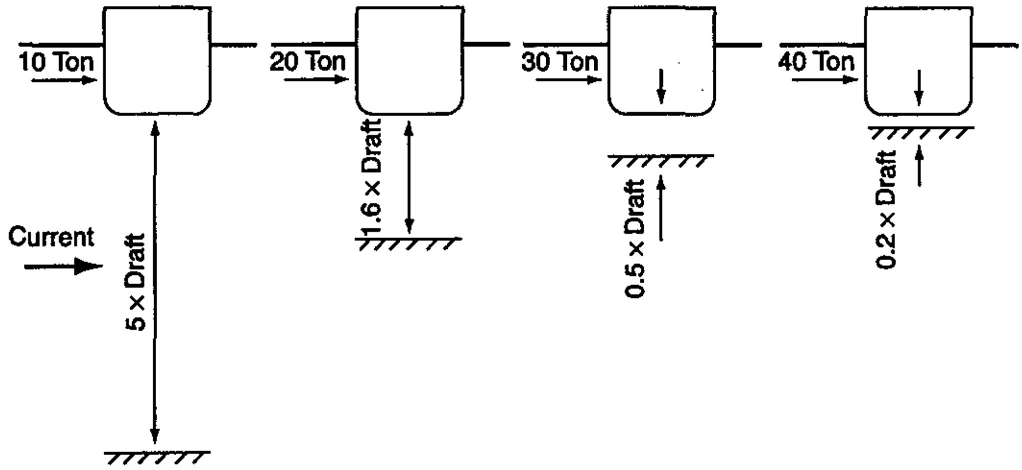


Fig. 2.14. The effect of underkeel clearance on the current force

In general the current forces on a ship due to velocity and direction follow a pattern similar to that for wind forces. When evaluating the mooring arrangement, the current forces must be added to the wind forces.

The current forces are complicated compared with the wind forces due to the significant effect of clearance beneath the keel. The effect of underkeel clearance on the current force is shown, in principle, in Fig. 2.14 by the increase in current force due to reduced underkeel clearance. Therefore, one shall always try to orientate the berth front parallel to the main current direction. Even a small current angle off the ship's longitudinal axis can create a transverse force, which must be evaluated during the evaluation of the mooring system.

2.4.2 Different current standards and recommendations

Below, some different methods and national standards and regulations methods for calculations of the current forces are shown and compared against each other. It should be noted that these standards and regulations should only be used as a guide to the magnitude of the forces on the ship. If more accurate current force calculations are needed these should be established by model testing in a hydraulic institute.

A **general standard formula** for calculation of the current forces on a ship:

Current pressure or force on a moored ship would be in kN:

$$P_c = C_C \times \gamma_w \times A_c \times \left(\frac{V_c^2}{2g}\right)$$

where C_C is the factor for calculation of the transverse and longitudinal forces. The magnitude of C_C depends to a large extent on the shape of the ship and the water depth along the front of the berth structure.

Factors for calculation of the **transverse** current force:

(a) For deep-water depth, gives $C_C = 1.0–1.5$
(b) Water depth/design ship draught $= 2$, gives $C_C = 2.0$
(c) Water depth/design ship draught $= 1.5$, gives $C_C = 3.0$
(d) Water depth/design ship draught $= 1.1$, gives $C_C = 5.0$
(e) Water depth/design ship draught nearly $= 1$, gives $C_C = 6.0$

Factors for calculation of the **longitudinal** current force varies from 0.2 for deep water to 0.6 for water depth to design ship draught nearly 1.

γ_w = specific gravity of seawater 10.34 kN/m^3
A_c = area of ship's underwater part projected on a plane perpendicular to the direction of the current
V_c = velocity of the current in m/s
g = acceleration of gravity 9.81 m/sec^2.

The Spanish Standard ROM 0.2-90, recommends that the current forces or pressure on a ship will be:

The lateral or **transverse** current force in kN:

$$F_{tcurrent} = F_{TC} + F'_{TC}$$

where the lateral or transverse current force in kN on the ship from pressure is:

$$F_{TC} = C_{TC} \times \gamma_w \times A_{LC} \times \sin\alpha \times \frac{V_c^2}{2g} \times 10$$

and the lateral or transverse current force in kN on the ship from drag is:

$$F'_{TC} = C_R \times \gamma_w \times A'_{TC} \times \sin^2\alpha \times \frac{V_C^2}{2g} \times 10$$

where $A_{LC} = L_{bp} \times D$ and $A'_{TC} = (L_{bp} + 2 \times D) \times B$.

The longitudinal current force in kN:

$$F_{lcurrent} = F_{LC} + F'_{LC}$$

where the **longitudinal** current force on the ship from pressure is:

$$F_{LC} = \pm\left(C_{LC} \times \gamma_w \times A_{TC} \times \frac{V_C^2}{2g} \times 10\right)$$

and the longitudinal current force on the ship from drag is:

$$F'_{LC} = C_R \times \gamma_w \times A'_{LC} \times \cos^2 \alpha \times \frac{V_C^2}{2g} \times 10$$

where $A_{TC} = B \times D$ and $A'_{LC} = (B + 2D) \times L_{bp}$

A_{LC} = the ship's submerged longitudinal projected area exposed to current
A'_{TC} = the ship's submerged transverse projected wetted area
A_{TC} = the ship's submerged transverse projected area exposed to current
A'_{LC} = the ship's submerged longitudinal projected wetted area
L_{bp} = the length between perpendiculars
D = the ship draught
B = the ship beam
γ_w = 1.03 t/m^3 for salt water and 1.00 for freshwater
g = the acceleration of gravity 9.81 m/sec^2
α = the angle between the longitudinal axis of the ship and the current direction considered from the bow
C_{TC} = the factor for calculating the transverse current force depending on the water depth/design ship draught as shown in Table 2.10
Water depth/design ship draught = 6 gives $C_{TC} = 1.0$
Water depth/design ship draught = 3 gives $C_{TC} = 1.7$
Water depth/design ship draught = 2 gives $C_{TC} = 2.7$
Water depth/design ship draught = 1.5 gives $C_{TC} = 3.8$
Water depth/design ship draught = 1.1 gives $C_{TC} = 5.7$

Table 2.10. Current coefficient
Spanish Standard ROM 0.2-90

Angle from bow to stern	Depth correction $D_w/D_d = 1.1$			Depth correction $D_w/D_d = 1.5$			Depth correction $D_w/D_d = 2.0$		
	C_{TC}	C_{LC}	C_R	C_{TC}	C_{LC}	C_R	C_{TC}	C_{LC}	C_R
0°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004
15°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004
30°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004
45°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004
60°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004
75°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004
90°	5.70	0.60	0.004	3.80	0.60	0.004	2.70	0.60	0.004

Table 2.10. Continued

British Standard BS 6349: Part 1: 2000   Transverse very large tankers

Angle from bow to stern	Depth correction $D_w/D_d = 1.1$			Depth correction $D_w/D_d = 1.5$			Depth correction $D_w/D_d = 2.0$		
	$C_{TCforward}$	C_{TCaft}	C_{CT}	$C_{TCforward}$	C_{TCaft}	C_{CT}	$C_{TCforward}$	C_{TCaft}	C_{CT}
0°	0.00	0.00	10.00	0.00	0.00	5.00	0.00	0.00	2.00
15°	0.50	0.20	8.00	0.50	0.20	4.50	0.50	0.20	1.70
30°	0.80	0.40	6.00	0.80	0.40	3.80	0.80	0.40	1.70
45°	1.20	0.70	5.00	1.20	0.70	3.40	1.20	0.70	1.60
60°	1.40	1.00	4.70	1.40	1.00	3.00	1.40	1.00	1.60
75°	1.50	1.30	4.80	1.50	1.30	2.90	1.50	1.30	1.70
90°	1.40	1.60	4.90	1.40	1.60	2.90	1.40	1.60	1.70
105°	1.20	1.70	4.70	1.20	1.70	2.90	1.20	1.70	1.70
120°	0.90	1.60	4.50	0.90	1.60	3.10	0.90	1.60	1.70
135°	0.60	1.40	4.50	0.60	1.40	3.60	0.60	1.40	1.70
150°	0.30	1.00	5.00	0.30	1.00	4.00	0.30	1.00	1.80
165°	0.10	0.60	6.50	0.10	0.60	4.40	0.10	0.60	2.00
180°	0.00	0.00	9.00	0.00	0.00	5.00	0.00	0.00	2.20

Longitudinal

	C_{LC}	C_{CL}	C_{LC}	C_{CL}	C_{LC}	C_{CL}
0°	0.40	1.70	0.40	1.40	0.40	1.20
15°	0.20	1.70	0.20	1.40	0.20	1.20
30°	−0.20	1.70	−0.20	1.40	−0.20	1.20
45°	−0.40	1.70	−0.40	1.40	−0.40	1.20
60°	−0.20	1.70	−0.20	1.40	−0.20	1.20
75°	0.20	1.70	0.20	1.40	0.20	1.20
90°	0.00	1.70	0.00	1.40	0.00	1.20
105°	0.10	1.70	0.10	1.40	0.10	1.20
120°	0.20	1.70	0.20	1.40	0.20	1.20
135°	0.20	1.70	0.20	1.40	0.20	1.20
150°	−0.10	1.70	−0.10	1.40	−0.10	1.20
165°	−0.40	1.70	−0.40	1.40	−0.40	1.20
180°	−0.50	1.70	−0.50	1.40	−0.50	1.20

C_{LC} = the factor for calculation the longitudinal current force due to the geometry of the ship's bow varying between 0.2 and 0.6. For conventional bows (bulb) equal 0.6 as shown in Table 2.10

C_R = the friction drag factor as shown in Table 2.10
For new ship = 0.001

Table 2.10. Continued

Oil Companies International Marine Forum, 1997

Angle from bow to stern	Depth correction $D_w/D_d = 1.1$ Loaded condition			Depth correction $D_w/D_d = 1.5$ Ballasted condition		
	C_{YFc}	C_{YAc}	C_{Xc}	C_{YFc}	C_{YAc}	C_{Xc}
0°	0.00	0.00	−0.04	0.00	0.00	−0.04
15°	0.73	0.22	0.04	0.08	0.03	−0.05
30°	1.05	0.47	0.05	0.19	0.10	−0.05
45°	1.05	0.60	−0.09	0.30	0.18	−0.03
60°	1.30	0.92	−0.11	0.39	0.27	−0.01
75°	1.40	1.30	−0.07	0.42	0.34	0.00
90°	1.40	1.54	0.00	0.39	0.40	0.01
105°	1.10	1.55	0.06	0.33	0.42	0.01
120°	0.90	1.40	0.09	0.26	0.39	0.02
135°	0.60	1.20	0.09	0.18	0.32	0.04
150°	0.26	0.95	0.04	0.11	0.20	0.06
165°	0.15	0.70	−0.01	0.04	0.09	0.06
180°	0.00	0.00	0.04	0.00	0.00	0.06

For ship in service = 0.004

V_C = the mean velocity of the current at an interval of 1 min at a depth of 50 per cent of the ship draft in m/s
Lacking of defined operating criteria, the following shall be adopted as limiting permanence velocities:
Cross current: $0° < \alpha < 180°$ $V_C = 1$ m/s (2 knots)
Bow or stern current: $\alpha = 0°$ and $\alpha = 180°$ $V_C = 2.5$ m/s (5 knots)
For mooring calculations under normal operation conditions, loading and unloading the design velocities shall be:
Cross current: $0° < \alpha < 180°$ $V_C = 1$ m/s (2 knots).
Bow or stern current: $\alpha = 0°$ and $\alpha = 180°$ $V_C = 1.5$ m/s (3 knots).

The British Standard BS 6349: Part 1: 2000, recommends that the current forces or pressure on a ship will be:

The lateral or transverse current force in kN:

$$F_{Tcurrent} = (C_{TCforward} + C_{TCaft}) \times C_{CT} \times \gamma \times L_{bp} \times d_m \times \frac{V_C^2}{10\,000}$$

The **longitudinal** current force in kN:

$$F_{Lcurrent} = C_{LC} \times C_{CL} \times \gamma \times L_{bp} \times d_m \times \frac{V_C^2}{10\,000}$$

where

$C_{TCforward}$ = the transverse forward current force coefficient depending on the angle of current as shown in Table 2.10
C_{TCaft} = the transverse aft current force coefficient depending on the angle of current as shown in Table 2.10
C_{CT} = the depth correction factor for transverse or lateral current forces as shown in Table 2.10
C_{LC} = the longitudinal current force coefficient as shown in Table 2.10
C_{CL} = the depth correction factor for longitudinal current forces as shown in Table 2.10
γ = the density of water taken as $1025\,\text{kg/m}^3$ for seawater and $1000\,\text{kg/m}^3$ for fresh water
L_{bp} = the length between perpendiculars
d_m = the mean draught of the ship
V_C = the average current velocity in m/s over the mean depth of the ship.

Oil Companies International Marine Forum 1997, recommends that the current force or pressure on a ship will be:

The **transverse** force:

$$F_{Ycurrent} = F_{YFcurrent} + F_{YAcurrent}$$

where the forward lateral or transverse current force on the ship is:

$$F_{YFcurrent} = C_{YFc} \times \gamma_w \times D \times L_{bp} \times \frac{V_c^2}{2012}$$

and the aft lateral or transverse current force on the ship is:

$$F_{YAcurrent} = C_{YAc} \times \gamma_w \times D \times L_{bp} \times \frac{V_c^2}{2012}$$

The **longitudinal** current force:

$$F_{XC} = C_{XC} \times \gamma_w \times D \times L_{bp} \times \frac{V_c^2}{2012}$$

where

C_{YFc} = the lateral or transverse forward current force coefficient depending on the angle of current and underkeel clearance as shown in Table 2.10
C_{YAc} = the lateral aft current force coefficient depending on the angle of current and underkeel clearance as shown in Table 2.10
C_{Xc} = the longitudinal current force coefficient depending of the angle of current and underkeel clearance as shown in Table 2.10
γ_w = the density of water taken as 1025 kg/m^3 for seawater
L_{bp} = the length between perpendiculars
D = the mean draught of the ship
V_C = the average current velocity in m/s acting over the mean depth of the ship
2012 = the conversion factor for velocity in knot/s to m/s.

The *Oil Companies International Marine Forum/Society of International Gas Tanker and Terminal Operators Ltd, 1995*, do not have any recommendations for calculating the current force or pressure on a tanker.

Comparison between the different current standards and recommendations
For a comparison between the different current standards and recommendations, the different current force coefficients due to the ratio of the water depth D_w and the ship's draft D_d and due to the effect of underkeel clearance on the current forces, are shown, rounded up, in Table 2.10.

Example of current forces on ship
Based on the above different current standards and recommendations' formulas and the different current force coefficients in Table 2.11, an example for comparison of the different current forces based on the different standards in kN acting longitudinal (0° against the bow), lateral or transverse (90° to the ship longitudinal axis) and longitudinal against the stern (180° against the stern, only the British Standard and the OCIMF 1997 give coefficients for this case) for different current speeds in m/sec for ships in loaded condition and with a 95 per cent

Table 2.11. Comparison of current forces on ship with different underwater clearance

Current speed in m/sec	General formula			Spanish Standard ROM 0.2-90			British Standard BS 6349 Part 1: 2000			OCIMF 1997		
	Forces in kN			Forces in kN			Forces in kN			Forces in kN		
	0°	90°	180°	0°	90°	180°	0°	90°	180°	0°	90°	180°
Underwater clearance $D_w/D_d = 1.1$												
1.0	360	18 600	–	432	21 167	–	492	10 635	615	126	10 572	126
Underwater clearance $D_w/D_d = 1.5$												
1.0	240	11 160	–	432	14 126	–	405	6294	506	–	–	–
Underwater clearance $D_w/D_d = 2.0$												
1.0	120	4460	–	432	10 050	–	347	3690	434	–	–	–

confidence limit after Chapter 20, are shown in Table 2.11 with the following ship dimensions:

(a) Oil tanker 200 000 dwt.
(b) $L_{OA} = 350\,\text{m}$, $L_{BP} = 346\,\text{m}$, $B = 56.2\,\text{m}$, $D_d = 20.4\,\text{m}$.

As can be seen from the current forces on a ship based on the different standards and recommendations, the current forces vary considerably between the different current standards and recommendations, than there are in wind forces between the different wind standards and recommendations. The main reason for the differences seems to be the different conservative attitudes to the design of current forces based on the empirical factors that should include all possible cases in terms of type, shape and size of the different ships, etc. The above methods for calculation should therefore only be used as a guide to the magnitude of the forces. Since the current forces can vary considerably, one should therefore, in order to obtain the most accurate forces for all important berth structures, do current testing of scale models in a hydraulic institute.

2.5 Ice forces

The study of forces acting on berth structures due to the formation of ice in the harbour basin has so far not been given high priority. However, where berths, dolphins, bridge pillars, etc. are surrounded by a solid ice slab during the winter season, or are exposed to drift ice, one must take into consideration that both horizontal and vertical

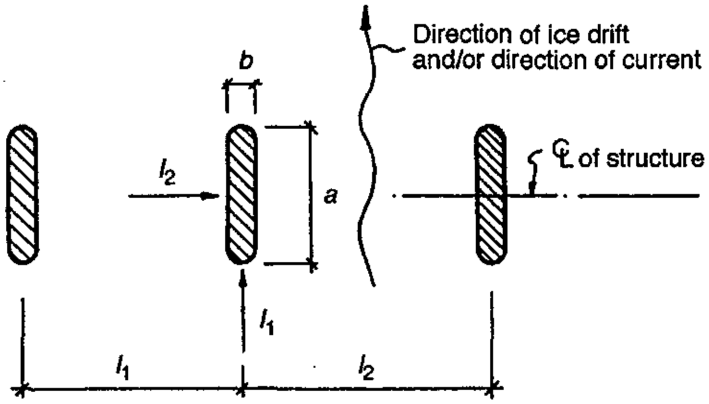


Fig. 2.15. Pressure from drift ice

ice forces can be of importance. One must also remember that water undergoes a volumetric expansion of about 8 per cent on freezing.

The magnitude of ice forces depends on the type and form of the structure and the properties of the ice. In general, ice formed in fresh water has higher strength and modulus of elasticity than ice formed in salt water. The unit weight of both freshwater and sea ice is about 9 kN/m^3. To state exact figures for the properties of strength is difficult because they depend to some extent on the conditions under which the ice was formed. The loads indicated below are meant as guidelines for an assessment of the magnitude of the ice forces.

Horizontal forces can act on a structure in both the transverse and longitudinal directions. Regarding transverse horizontal forces due to drift ice or drifting ice slab, reference is made to Fig. 2.15:

$$I_1 = i_1 \frac{(l_1 + l_2)}{2} \quad \text{or} \quad I_1 = i_1 \times X$$

where i_1 is the at rivers or berth structures under traffic is 10–20 kN tons/lin m; at sounds, fjords and narrow bays is 30 kN/lin m; at places heavily exposed to ice is 50–100 kN/lin m. X is the width of the ice sheet (floe) in m.

Alternatively:

$$I_1 = m \times s_t \times b \times d$$

where m is the form factor, 0.8 for circular forward edge, 1.0 for straight-lined forward edge. s_t is the compressive strength of the ice, 1500 kN/sq m in freshwater, 1000 kN/sq m in salt water, d is the thickness of the ice = 0.8 × maximum ice thickness.

Research has shown that the maximum velocity of moving ice under the influence of a steady wind will not be more than 3 per cent of the wind velocity.

Regarding longitudinal horizontal forces due to *inter alia* thermal expansion pressure, reference is also made to Fig. 2.15:

$$I_2 = i_2 \times a$$

where i_2 if there is open water on the other side of a pillar = 100–300 kN/lin m and if there is firm ice slab on all sides is 1/4 of the above values = 25 to 75 kN/lin m.

Alternatively:

$$I_2 = 0.2 \times I_1$$

At berth structures being called at frequently and at structures in waters with a great tidal variation where the formation of ice is hampered, horizontal ice loading usually causes no problem if the structure has been designed for an ice loading of 10–20 kN/lin m of berth front.

Experiments have shown that the horizontal force due to ice against a pillar with 45° sloping sides can be reduced to $\frac{1}{3}$ of the force acting against a structure with vertical sides.

Structures that are firmly frozen in may be subjected to vertical forces by the ice during tidal variations. This can be a problem for light structures having too small a dead weight to prevent the ice from lifting them; for instance light piers on timber piles. As a general guideline, the vertical lifting ice forces on a pile are inversely dependent on the magnitude of the tidal variation, i.e. the larger the tidal range the less will be the lifting forces, owing to the greater difficulty for the ice to freeze to a pile when there is a large tidal variation.

As a guide in the design of structures exposed to ice, Mr Løfquist of Sweden has suggested the application of the design forces shown in Figs 2.16 and 2.17. The figures are based on ice with a bending

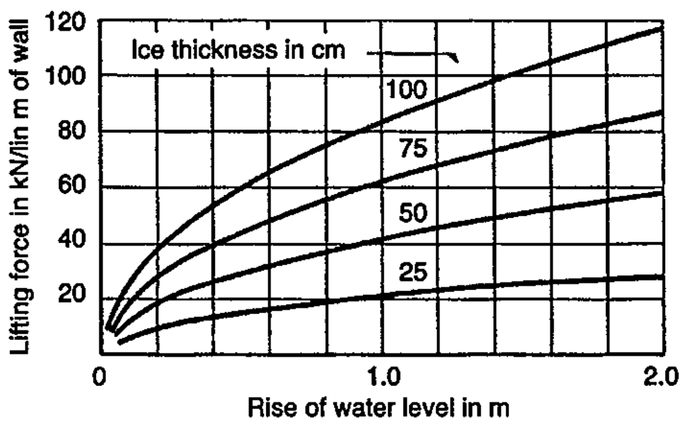


Fig. 2.16. Lifting forces on wall

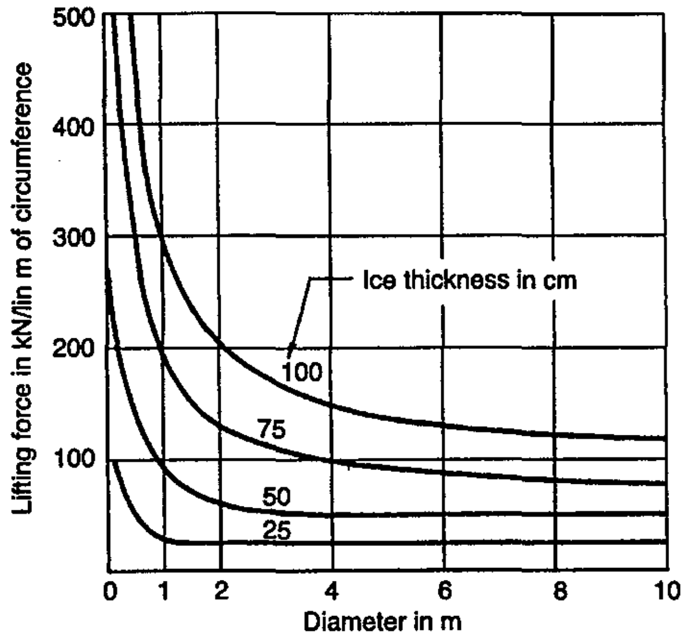


Fig. 2.17. Lifting forces on column

strength equal to 2000 kN/m^2. The lifting force/lin m of straight-lined wall and the lifting force/lin m of the circumference of a circular element column are shown respectively. For square-shaped elements the inscribed circle is to be used to determine the perimeter length.

In cases where there are severe problems in connection with the formation of ice in the port basin, the installation of one or more compressed-air bubbling plants to keep the water open would probably prove more economical than designing the berth structures for greater loadings. Compressed-air bubbler de-icing is an effective ice suppression and control method. The air bubbles move warmer bottom water upwards to melt the underside of the surface ice.

As a rule of thumb the bearing capacity of good ice in relation to the thickness of the ice is as shown below:

Man on foot	5 cm
Motor cycle	10 cm
Small car	20 cm
Tractor	30 cm
Truck (2.5 ton)	40 cm
Aeroplane (9 ton)	50 cm

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3

Channels and harbour basins

3.1 Channels and waterways

3.1.1 General

From a general point of view, channels or waterways can logically be classified into the following four groups:

(a) Group A: main traffic arteries which have satisfactory day and night navigational aids and where given depths are guaranteed.
(b) Group B: same as group A, but with navigational aids for day navigation only.
(c) Group C: important routes, which may have navigational aids and where depths are checked by regular surveys, but not guaranteed.
(d) Group D: local routes which have no navigational aids and where only estimates of depths are given.

Channels or waterways can again be subdivided into unrestricted, semi-restricted and fully restricted channels:

(a) **Unrestricted channels** are channels or waterways in shallow water of width at least 10–15 times the beam of the largest ship using the channel, but without any dredging.
(b) **Semi-restricted channels** are dredged channels in shallow water. See Fig. 3.1.
(c) **Fully restricted channels** are channels where the entire channel area is dredged, as shown in Fig. 3.1. In general the layout and the alignment of the channels should be such that the channel can be navigated with reasonable safety according to into which group the channel is classified taking account of tide, current, prevailing wind and wave action.

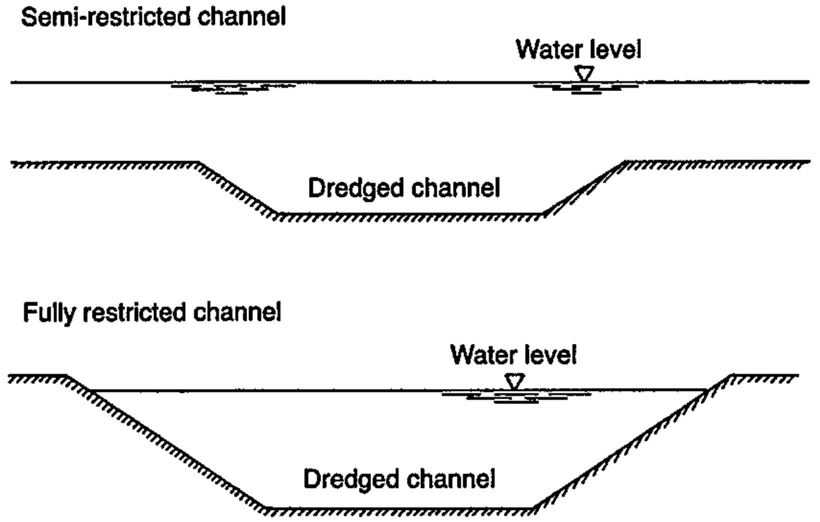


Fig. 3.1. Semi- and fully restricted channels

If possible, the angle between the resultant effect due to the prevailing wind direction and current and the channel axis should be a minimum. The angles of deflection and the number of curves in the channel should also be kept to a minimum.

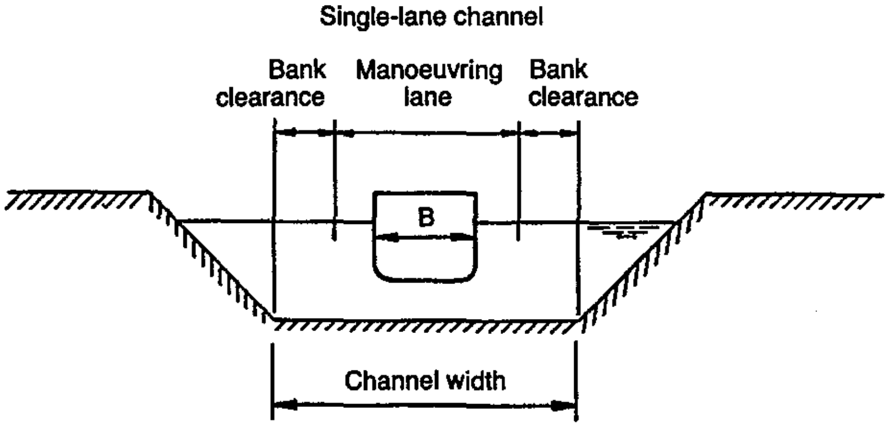
An example of a fully restricted channel or canal is the **Panama Canal**, where the maximum dimensions for ships using the canal are: the overall length 294 m; the width of beam 32.31 m; and the maximum draft 12 m.

The channels should preferably be located in areas of maximum natural water depth to reduce the cost of initial and maintenance dredging. Areas which are exposed to excessive siltation and littoral drift should be avoided if possible. However, to maintain a minimum depth, as shown on navigational charts, maintenance dredging is usually necessary. The volumes to be dredged can vary widely from place to place, depending on the extent of the site, its location and other natural influences such as tides, current and weather conditions.

3.1.2 Straight channel

The minimum width of a straight channel will primarily depend on the size and manoeuvrability of the ships navigating the channel and the effects of wind and current. The channel width is divided into three zones or lanes, as shown in Fig. 3.2 for one-way and two-way traffic:

(a) the manoeuvring lane
(b) the bank clearance lane
(c) the ship clearance lane.



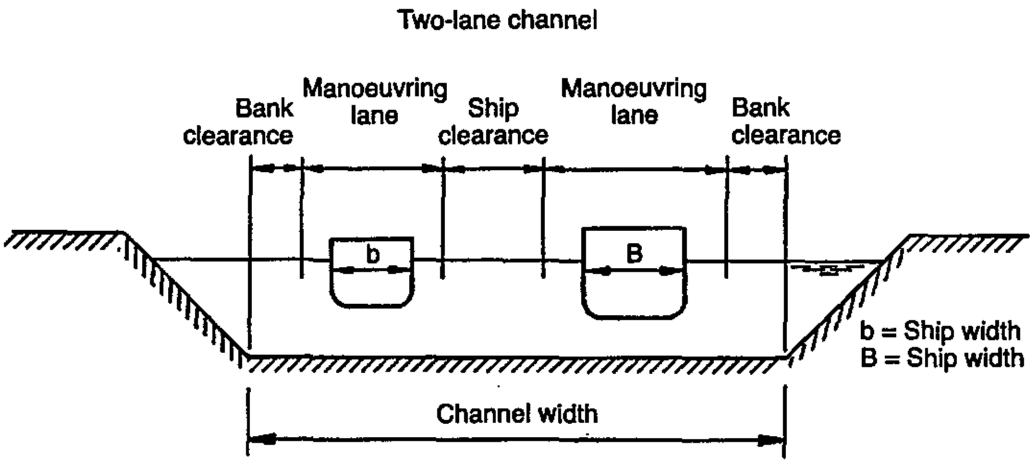


Fig. 3.2. Channel width

The width of a restricted channel should be measured at the bottom of the dredged bed and should be the sum of the lanes.

The width of the **manoeuvring lane** will generally vary from 1.6–2.0 times the beam of the largest ship using the channel, depending on wind, current and the manoeuvrability of the ship. The very high superstructures on containerships, car carriers, passenger ships and tankers in ballast present considerable windage area and may therefore require more channel width than their beam would suggest.

Allowance for yaw of the ship must be made if the channel is exposed to cross current and/or winds. The angle of yaw can be between 5°–10°. For a large ship, an angle of yaw of 5° can add an extra width, equivalent to half the beam, to the manoeuvring lane.

Ships displaced from the channel centreline towards the banks of the channel will experience a bank suction effect due to the asymmetrical flow of water round the ship and this will cause a yawing movement. To counteract this effect on the ship an additional **bank clearance**

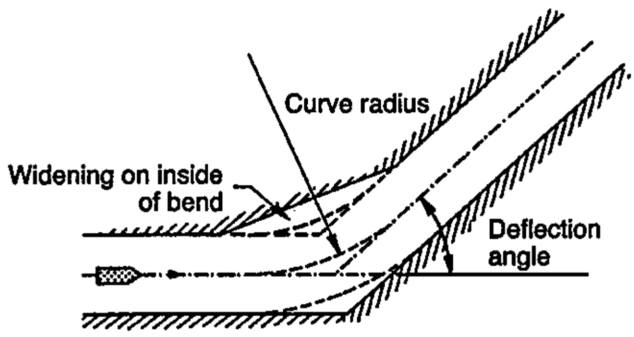


Fig. 3.3. Channel curve

width usually between 1.0–2.0 times the beam of the largest ship must be added. A steep-sided channel section produces more bank suction than a channel with a trapezoidal section. Bank suction also increases when the underkeel clearance decreases.

To avoid excessive interaction between two ships travelling past one another, either in the same or in the opposite direction in a two-lane channel, it is necessary to separate the two manoeuvring lanes by a ship clearance lane. To minimize the suction and repulsion forces between the ships, a **clearance lane** equal to a minimum 30 m, or the beam of the largest ship, should be provided.

The recommended total channel bottom width for **single-lane** channels should be 3.6–6 times the beam of the design ship depending on the sea and wind conditions. For oil and gas tankers a minimum bottom width should be 5 times the beam of the design ship. For a **two-lane** channel the total channel width will vary between 6.2–9 times the beam of the design ship.

3.1.3 Channels with curves

As a general rule, curves and sharp turns in a channel should be avoided if possible. Where curves are unavoidable, the minimum width of the channel in a curve should be larger than in a straight channel due to the additional manoeuvring width required, because the ship will deviate more from her course in a bend than in a straight section. In Fig. 3.3 definitions of the curve radius and deflection angle are illustrated.

In practice, if the deflection angle of the curve is larger than 10°, the channel should be widened. It is generally accepted that a widening of the inside of the curve or bend is the most suitable manner to improve safe navigation in a curve. Depending on the manoeuvrability of the ship and the radius of the bend, the width of the manoeuvring lane should be increased from around 2.0 times the beam of the largest ship in a straight channel to around 4.0 times the beam of the largest ship in curved channels.

In the past it was accepted that for ships without tugboat assistance, the minimum curve radius should not be less than 3 times the length of the design ship for a deflection angle of the curve up to 25°. Between 25° and 35° the minimum curve radius should be 5 times the length of the design ship. For 35° and more the curve radius should be 10 times the length of the design ship. If the curves must have smaller radii than mentioned above, the channel should be suitably widened. More recent proposals suggest that the minimum curve radius should be in the range of 8–10 times the length of the design ship, without being related to the angle of deflection.

If more than one curve is necessary, a straight section equal to at least 5 times the length of the design ship or 1000 m, whichever is greater, should be provided between the two consecutive curves.

3.2 Harbour basin

3.2.1 General

The harbour basin can be defined as the protected water area, which should provide safe and suitable accommodation for ships. Harbours can be classified as natural, semi-natural or artificial. Harbours have different functions, such as commercial (municipal or privately owned) harbours, refuge harbours, military harbours, oil harbours, etc.

Inside the harbour entrance, the harbour area should be allocated different functions such as berthing or turning area. If the harbour receives a wide range of ships, it should for economic reasons be divided into at least two zones, one for the larger and one for the smaller ships. The smaller ships should be located in the inner and shallower part of the harbour. Berths for hazardous cargoes like oil and gas should be located at a safe distance and clearance from other berths. These activities should typically be located in isolated areas in the outer end and on the lee side of the harbour basin.

3.2.2 Entrance

The harbour entrance should, if possible, be located on the lee side of the harbour. If it must be located on the windward end of the harbour, adequate overlap of the breakwaters should be provided so

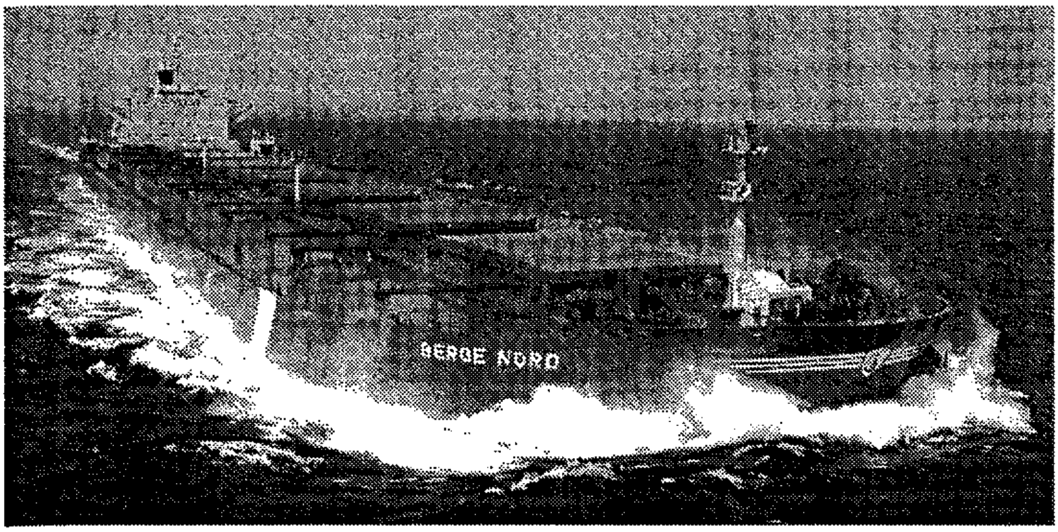


Fig. 3.4. A loaded ore tanker. Photo by Bergesen DY, Norway

that the ship will have passed through the restricted entrance and be free to turn with the wind before it is hit broadside by the waves. Due to this overlap of the breakwaters the interior of the harbour will be protected from the waves. Accordingly, in order to reduce the wave height within the harbour, and to prevent strong currents, the entrance should be no wider than necessary to provide safe navigation.

The entrance width measured at the design depth will depend on the degree of wave protection required inside the harbour, the navigational requirements due to the size of ship, density of traffic, depth of water and the current velocity when the tide is coming in or going out. Generally the width of the harbour entrance should be between 0.7–1.0 times the length of the design ship.

The maximum current velocity through the harbour entrance should not exceed approximately 1.5 m/s or 3 knots if possible. If the current velocity exceeds this value, the channel cross-section should be adjusted.

3.2.3 Stopping distance

The stopping distance of a ship will depend on factors such as ship speed, the displacement and shape of the hull, and horsepower ratio. The following stopping distances, as a rough guideline, are assumed to be sufficient to bring the ship to a complete halt. For ships in ballast, 3–5 times the ship's length is required. For a loaded ship, as shown in Fig. 3.4, 7 to 8 times the ship's length is required. In harbours where the entrance is exposed to weather, the stopping distance should typically be reckoned from the beginning of the protected area to the centre of the turning basin.

3.2.4 Turning area

The turning area or basin should usually be in the central area of the harbour basin. The size of the turning area will be a function of manoeuvrability and of the length of the ship using the area. It will also depend on the time permitted for the execution of the turning manoeuvre. The area should be protected from waves and strong winds. One should remember that ships in ballast have decreased turning performance.

The following minimum diameters of the turning area are generally accepted. The minimum diameter where the ship turns by going ahead and without use of bow thrusters and/or tugboat assistance, should be approximately 4 times the length of the ship. Where the ship has tugboat assistance, the turning diameter could be 2 times the length of the ship. Under very good conditions these diameters might be reduced to 3 and 1.6 times the length respectively as a lower limit. With use of the main propeller and rudder and the bow thrusters, the turning diameter could be 1.5 times the length of the ship.

Where the ship is turned by warping around a dolphin or pier and usually with tugboat assistance under calm conditions, the turning diameter could be a minimum 1.2 times the length of the ship.

3.2.5 Berthing area

The size of the berthing area and the berth will depend upon the dimensions of the largest ship and the number of ships that will use the harbour. The berth layout will be affected by many factors such as the size of the harbour basin for manoeuvring, satisfactory arrivals and departures of ships to and from the berth, whether or not the ships are equipped with bow rudder and bow thrusters, the availability of tugboats, and the direction and strength of wind, waves and currents.

If the berthing area in **front of the berth** has to be dredged, the size of the dredging area should be as shown in Fig. 3.5. The length of the dredged area should be for ships with tugboat assistance not less than 1.25 times the length of the largest ship to use the berth, and without tugboat assistance not less than 1.5 times the length. The width of a dredged tidal berth should be at least 1.25 times the beam of the largest ship to use the berth.

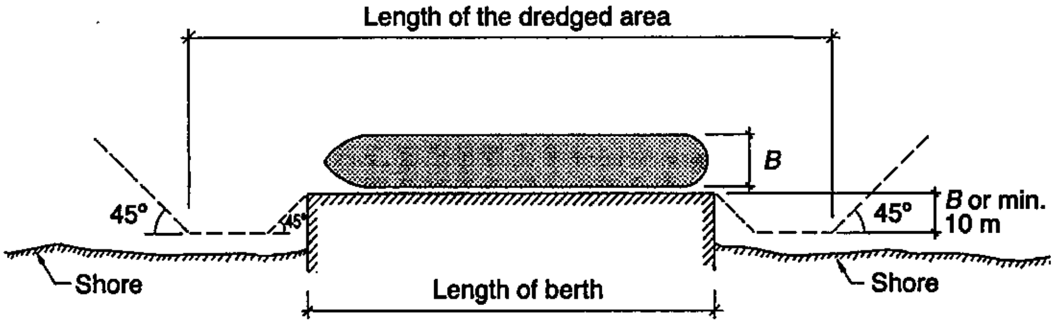


Fig. 3.5. Dredged area around a berth

Where more than one ship has to be accommodated along the berth, as shown in Fig. 3.6, a clearance length of at least 0.1 times the length of the largest ship should be provided between the adjacent ships. If the harbour basin is subjected to strong winds and tides the clearance should be increased to 0.2 times the length of the largest ship. A minimum distance of 15 m between the ship is commonly adopted.

Berths of the **finger pier** type, as shown in Fig. 3.7, will provide the greatest amount of berthing space per metre of shorefront. For a single-berth pier, the clear water area between two piers should be 2 times the beam of the largest ship plus 30 m to allow for tugboat assistance. For double-berth finger piers the clear water area between two double-berth piers should be 4 times the beam of the largest ship plus 50 m. The length of the finger pier for a single berth should, if possible, be the length of the ship plus 30–50 m. For very long single-berth piers, as shown in Fig. 3.8, the clear water area between the two piers should be 2 times the beam of the largest ship plus 50 m.

For harbour basins, as shown in Fig. 3.9, the width required to permit a ship to swing freely into a berth is 1.5 times the length of the ship for berths at 45°, and 2 times the length of the ship for berths at 90°.

The layout of berthing structures for **oil and gas** tankers is different from the berth layout for general cargo ships. The major components of an oil and gas berthing structure are as shown in Fig. 3.10 and include

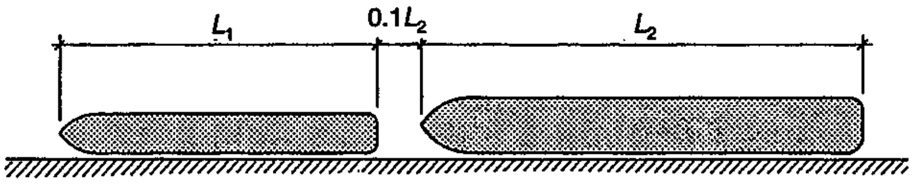


Fig. 3.6. Clearance between ships at berth

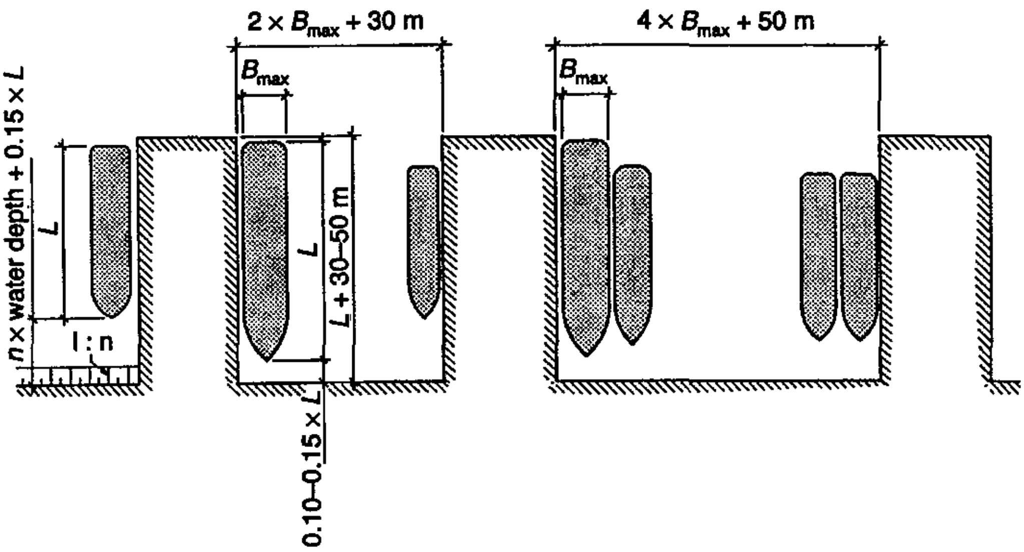


Fig. 3.7. Layout of single piers

the following elements: the mooring structures, the breasting structures, the loading platform and the access bridge with the pipeway. For the safety of the tanker and the tugboats it is important that there is enough manoeuvring space provided for the tugboats around the tanker during its berthing and mooring.

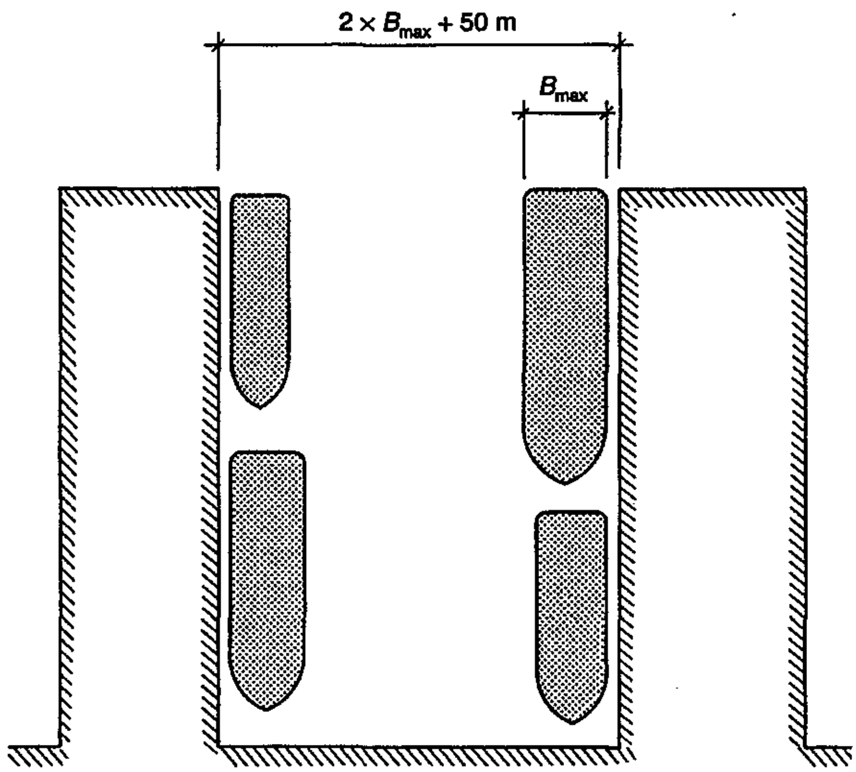


Fig. 3.8. Layout of long piers

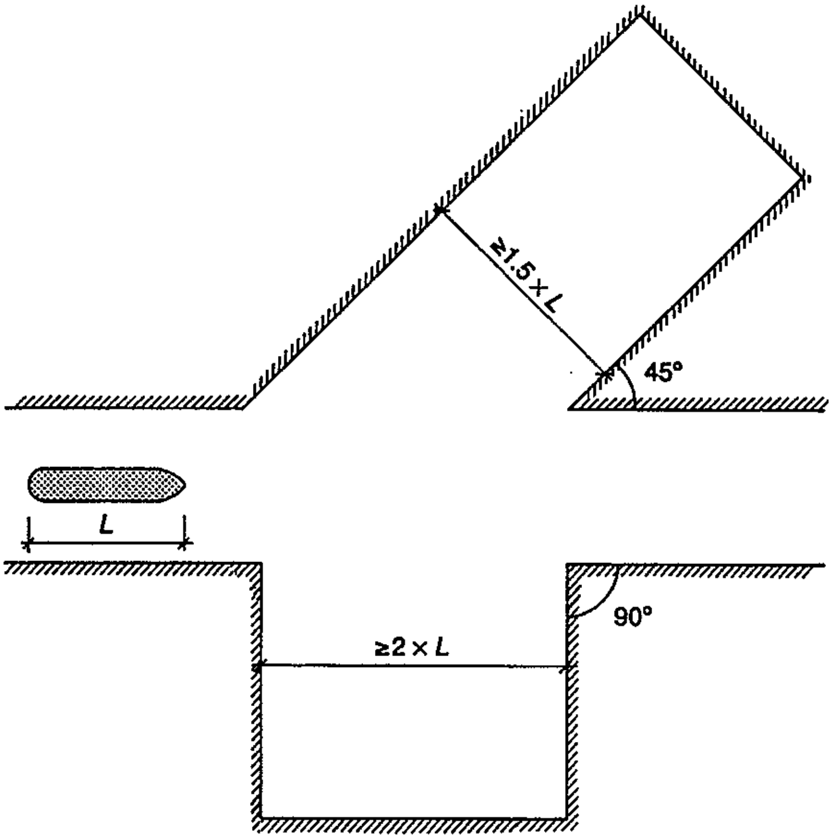


Fig. 3.9. Layout of berths

The berth should, if possible, be oriented so that the predominant wind, wave and current have the least effects on the operation of the berth. The berth should be so oriented that the mooring loads are as small as possible. Usually, this means aligning the berth axis with the current direction. Where the currents are weak, it is advisable to locate the berth parallel to the prevailing wind direction. Berths should not be broadside on to strong prevailing winds, waves and current.

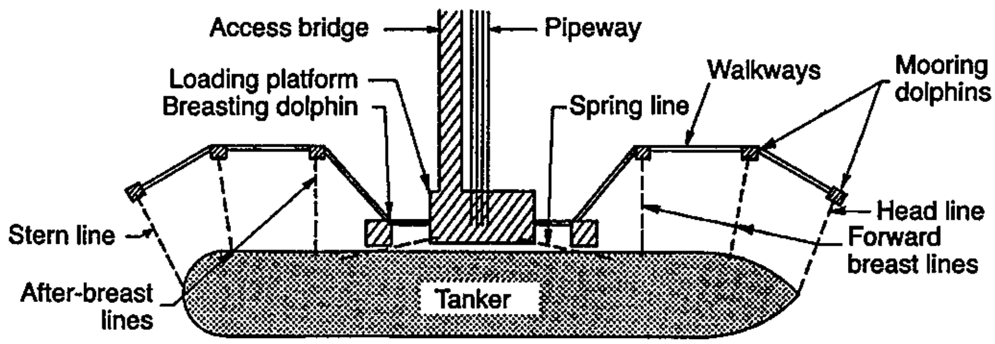


Fig. 3.10. Typical berth for tankers

The loading platform and the breasting structure can either be built as one structure or as two separate structures. The breasting structures should be designed to withstand the berthing impact from the tanker during berthing and from the wind, wave and current forces when moored. The mooring structures should be designed for the mooring and environmental forces.

To ensure contact with the parallel sides of the ship, the breasting **structures** should be set apart as described in Chapter 4. Walkways should be provided between the mooring structures and the central structures. Although one breasting structure on each side of the loading platform is adequate for safe berthing of a tanker, it is recommended that two breasting structures be provided on each side in case one of the breasting structures is damaged during berthing.

The **safety distance** between two moored tankers or a moored tanker and a passing ship, will depend upon the overall layout of the harbour, the number of tugboats assisting in the berthing or unberthing operation, the environmental conditions and the population in the area. The distances also vary from country to country depending on the safety philosophy in each country. The safety distance may be found to vary between the following ranges:

For **oil tankers**:

(a) The distance between two moored oil tankers may be from 30–100 m.
(b) The distance between a moored oil tanker and a passing ship may be from 50–150 m.

For **gas tankers**:

(a) The distance between two moored gas tankers may be from 50–150 m. For liquefied natural gas (LNG) tankers a minimum clearance of one ship's length between ships or 250–300 m is recommended. The distance between LNG tankers will also depend on the tugboat capacity when berthing and unberthing the tanker.
(b) The distance between a moored gas tanker and a passing ship may be from 60–250 m. For a LNG tanker it shall at least be 300 m.
(c) It is generally accepted that for liquefied petroleum gas (LPG) the distance may be at least 150 m to other installations and for LNG it should be at least 300 m.

A general fairway outside an oil and gas terminal, should preferably be outside the turning basin in front of the oil or gas berth, so that

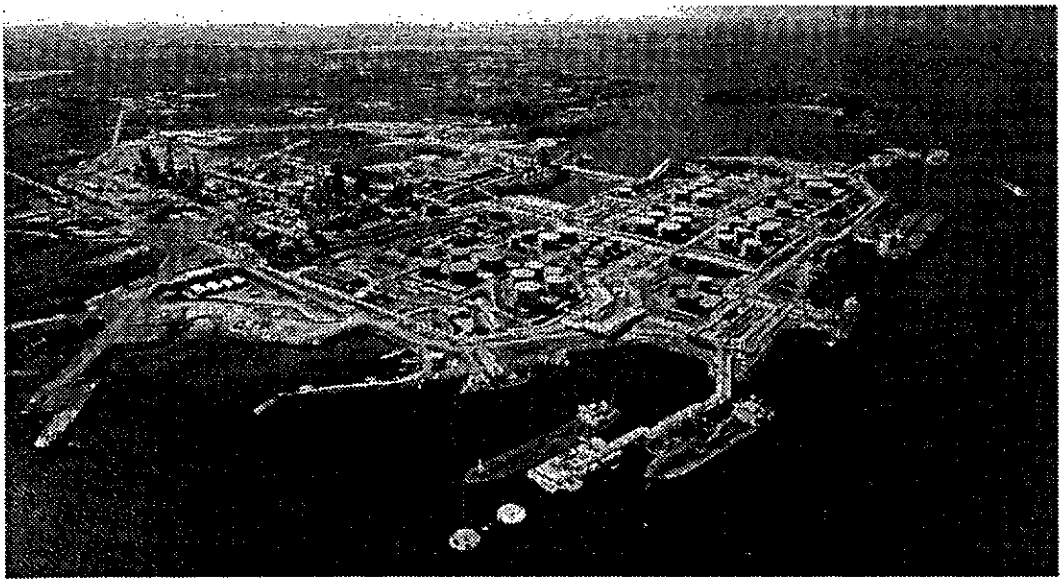


Fig. 3.11. Statoil, Mongstad Oil Terminal. Photo courtesy of Øyvind Hagen, Statoil, Mongstad, Norway

the berthing operation will not be disturbed by passing vessels in the fairway.

Due to safety and risk considerations, it is recommended that, for example, LNG terminals are placed in sheltered locations remote from other port activities so other ships do not pose a collision risk to a moored LNG tanker. Furthermore, if a large ship is passing close to a moored LNG this can cause surging along the gas berth with risk to the mooring lines of the gas tanker.

The **orientation** of oil and gas berths should be chosen to provide the best possible manoeuvring conditions for normal berthing and unberthing as well as emergency departure, as can be seen from Fig. 3.11. Under calm-weather conditions tankers should preferably be able to depart without tugboat assistance, although this is not recommended as normal procedure.

At oil terminals, a portable collecting oil barrier should be placed around the oil tanker prior to loading or be able to be placed at very short notice in order to restrain any oil spillage. Equipment for collection and disposal of oil spills must also exist.

The general area requirements for **small craft** harbour berthing arrangements are shown in Figs 3.12, 3.13 and 3.14. The general measurements will vary as shown in the figures depending on the layout of the harbour. In places with large tidal variations, or where the harbour is exposed to wind and/or waves, the maximum figures

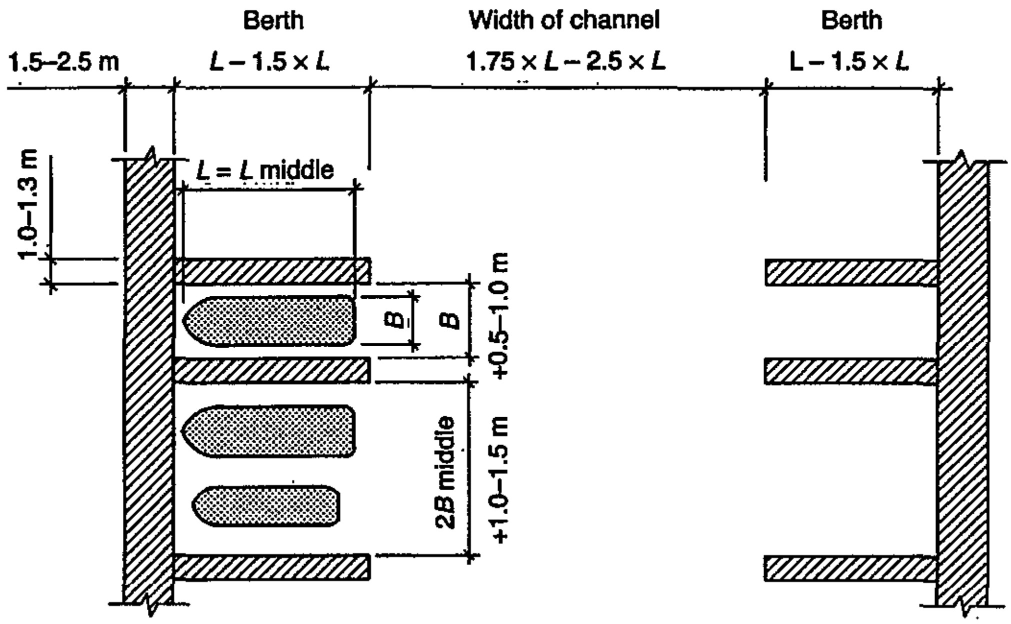


Fig. 3.12. General layout of small craft harbour berthing arrangement

must be used. The normal figure for the total water area required per boat will vary between 100 and 200 m^2 per boat.

No rules exist for the size of the berthing area for a **fishing port**, but widths of about 100–150 m and lengths of about 200–400 m are common in existing ports. For safety reasons and depending on the

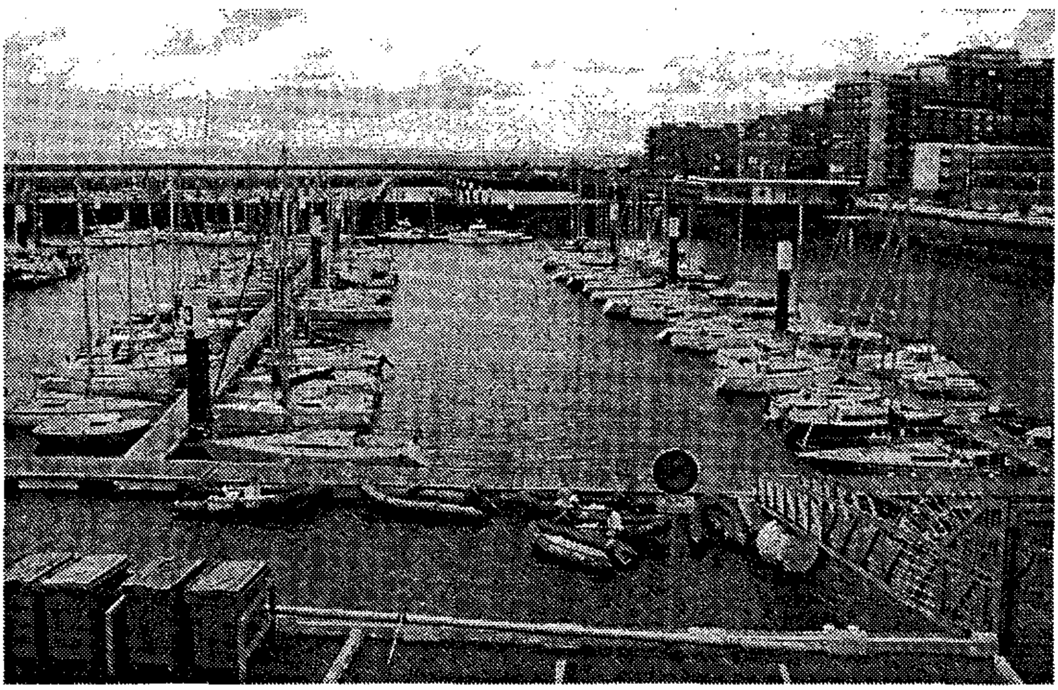


Fig. 3.13. Small craft harbour

Fig. 3.14. Small craft harbour

use of the port facilities, it is not desirable to have more than about three or four fishing ships berthing side by side along the berth.

3.3 Anchorage areas

The anchorage area is a place where ships may wait for their turn at berth, for more favourable weather conditions or be held back for quarantine inspection or other reasons. Sometimes special anchorage places are provided for ships carrying dangerous cargo, such as explosives.

The size of water area required for anchorages will therefore primarily depend on the number, type and size of ships, which require protection and the type of mooring systems available. The selection of the mooring system will depend on the size of ship, degree of exposure to weather, degree of restraint required and quality of the sea-bottom material (the anchor holding). As a general rule, the harbour should provide anchorage areas for small coastal ships while they are waiting for their turn to call at berth or for protection in bad weather, while larger ships may be required to anchor or ride out bad weather at open sea if necessary. The anchorage areas should be located in natural protected areas or be protected from waves by breakwaters and also be located near the main harbour areas, but out of the path of the main harbour traffic.

The water depth at an anchoring area should preferably not exceed approximately 50–60 m due to the length of the anchor chain of the ship. The bottom condition must not be too hard, otherwise the anchor will be dragged along the bottom and not dig into the sea bottom.

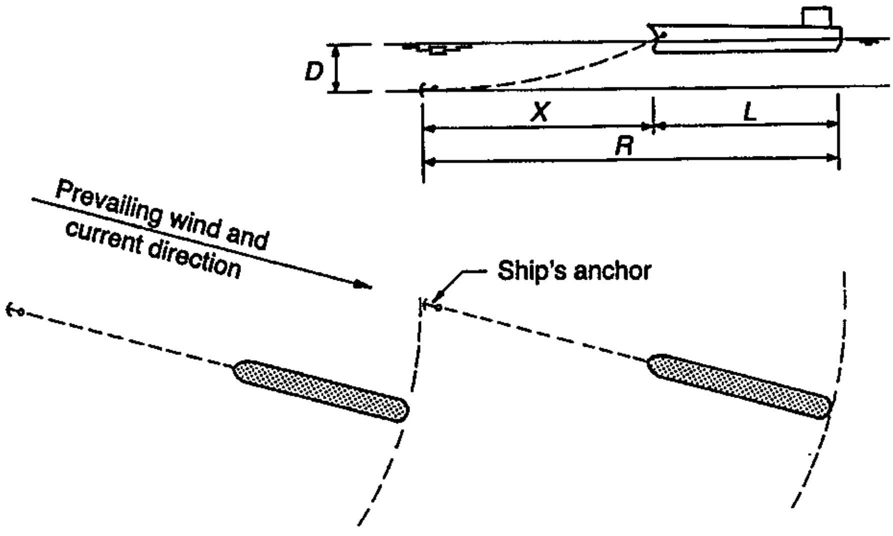


Fig. 3.15. Free-swinging mooring

When the ship is anchored the following should be adhered to in the anchoring procedures in addition to observing all port traffic and following the port's regulations:

(a) Maintain a 24-h bridge watch by a licensed deck officer monitoring the radio contacts.
(b) Make frequent control checks to ensure that the ship is not dragging the anchors.
(c) When the wind exceeds 20 m/s, put the propulsion plant on standby for the possibility of leaving the area.
(d) Provide a 15-min advance notice to the respective pilot station before heaving the anchors to get underway.

A ship may be moored either with its own anchors, to a buoy or group of buoys or by a combination of its own anchors and buoys. Mooring systems can, therefore, be divided into the free swinging systems and the multiple-point mooring systems.

When using the **free-swinging mooring** system, as shown in Fig. 3.15, the ship will swing on its anchor and be located generally parallel to the wind and current. The anchorage area shall, therefore, have a water area exceeding the area of a circle with the radius obtained from Table 3.1 and in accordance with the natural conditions, such as topography, sealed condition and exposure to weather:

The horizontal distance X in Fig. 3.15 will usually vary between approximately 6–10 times the water depth. The length of the anchor chain can be reduced for a single buoy by adding a deadweight near

Table 3.1. Approximate radius of anchorage area

Object of anchorage	Seabed soil or wind velocity	Radius in Fig. 3.15
Waiting or cargo handling	Good anchoring	$L + 6D$
	Bad anchoring	$L + 6D + 30\,\text{m}$
Mooring during	Wind velocity 20 m/s	$L + 3D + 90\,\text{m}$
	Wind velocity 30 m/s	$L + 4D + 145\,\text{m}$

the buoy, e.g. a concrete block for holding the chain between the buoy and the anchor down to the sea bottom.

In the **multiple-point mooring** system shown in Fig. 3.16, the ship is secured to a minimum of four mooring points and is thereby held in a more or less fixed position.

To obtain the maximum pullout or anchor resistance, the anchor chain must not be subjected to a pull angle of more than 3° above the horizontal near the anchor. The maximum pullout of the anchor, depending upon the soil condition, is about 7 to 8 times the weight of the anchor. The anchor weight for a 40 000 dwt ship is about 7 tons, and about 21 tons for a 200 000 dwt ship. If the pullout angle is 5° above the horizontal, the maximum pullout of the anchor is reduced by about 25 per cent, and if the angle is about 15°, the maximum pullout of the anchor will be reduced by about 50 per cent.

The anchorage area should have enough water area for the possibility of drift when releasing the anchor line of approximately 3 times the water depth. The underkeel clearance of the ship should never be less than approximately 3–4 m at the lowest astronomical tide (LAT).

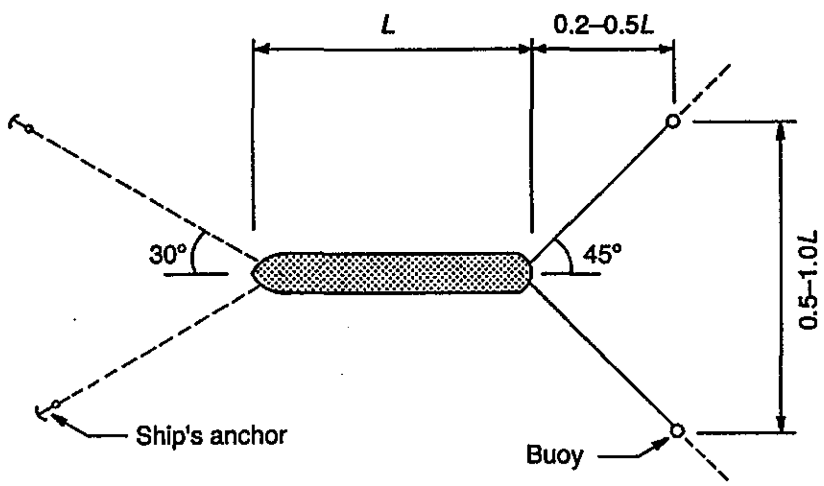


Fig. 3.16. Multiple-point mooring

As a rule of thumb, the length of the anchor chain of a ship is approximately 1.5 times the ship's length.

3.4 Grounding areas

In the case of serious damage to ships, emergency grounding areas along the approach route to or from the harbour should be available. It is very important to have the possibility of grounding an oil tanker, for example in a case where the tanker has been damaged to such an extent that there is a risk of it sinking and thus causing extensive oil pollution.

The water depth at the grounding area should be a little less than the draft of the ship. The sea bottom should preferably be even and soft. The manoeuvring should be as easy as possible since a damaged ship may have reduced manoeuvrability.

Further reading

British Standard BS 6349 (1988) *Maritime Structures. Part 2: Design of Quay Walls, Jetties and Dolphins*, London: BSI.

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Permanent International Association of Navigation Congresses (PIANC) (1997) *Approach Channels, a Guide for Design, Report of Working Group II-30.*

ROM 0.2-90 (1990) *Action in the Design of Maritime and Harbour Works*, Ministerio de Obras Publicas, Madrid, April.

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4

Berthing requirements

4.1 Operational conditions

4.1.1 General

In the evaluation of the operational conditions one should always bear in mind that manoeuvring a ship in confined shallow water, in close proximity to other ships such as in a navigational channel or inside a harbour, is entirely different from manoeuvring a ship in deep water on the open sea with infrequent and distant traffic.

The design and evaluation of the operational conditions shall be based on the following principles:

(a) The design shall be based on common and proven technology from similar harbour projects.
(b) The design shall be based on internationally acceptable standards and recommendations.

In the planning and evaluation of a proposed harbour site the collection of information on tide, current, wind, waves, etc. plays an essential role together with the hydrographic and topographical conditions. These factors are essential both for the safety of the ship during navigation and berthing operations as well as for cargo-handling operations. Together all these factors will determine the total operational availability of the harbour or the berth.

Generally ships are primarily designed for the open ocean, and therefore dramatic changes can occur in a ship's response characteristics in shallow water. Safe manoeuvring and berthing in confined water require an adequate design and layout of the navigational channels and harbour, and a proper understanding of the environmental forces which can act on a ship.

Ships that are difficult to handle in confined water, can be roughly divided into two groups:

(a) Large, deep-draft ships, such as fully loaded tankers with small underkeel clearance.
(b) Ships with very high superstructures, such as container ships, car carriers, passenger ships and tanker in ballast.

The environmental conditions are of prime importance for a marine terminal designed to accommodate ships of the very large crude carrier (VLCC) class. The wind forces on a VLCC tanker in ballast, or current on such a ship in a fully loaded condition, can give rise to considerable forces. Ships with lengths up to 350–400 m, as is the case of a VLCC tanker, can experience great variations of environmental forces along the ship's length.

For some of these parameters, long-term measurements are needed to establish statistical data as a basis for the preliminary and detailed design of the harbour. These measurements should, if possible, be made for at least one year in order to describe the seasonal variations. Measurements of sediment transport should be made under rough weather conditions in order to give reliable information, due to the fact that the processes of interest only occur under extreme current and wave conditions. All these measurements should be as close to the proposed harbour site as possible.

Proper planning of all the field investigations will require a good understanding of the characteristics of the proposed harbour site, and should always be undertaken by an experienced coastal and harbour engineer in order to get the most valuable and optimal information from the site investigations.

In the planning and evaluation of the approach to a terminal harbour site, the collection of data on tide, current, wind, waves, fog, etc., plays an essential role together with the hydrographic and topographical conditions. These factors are essential for the safety of vessels during navigation and berthing operations.

4.1.2 Tide

The tide consists of two components: astronomical effects and meteorological effects.

The astronomical tidal variation can be found in the **Admiralty Tide Tables**. The astronomical tidal day is the time of rotation of the earth with respect to the moon and the planets, and is approximately 24 h and 50 min. The ebb and flood tide are the falling and rising tide respectively.

The **chart datum** for harbour works is generally the **Lowest Astronomical Tide** (LAT) which is the lowest tide level that can be predicted to occur under average meteorological conditions. For the berth structures itself, and for land installations, the chart datum is usually referred to approximately the mean water level.

The following water levels should always be recorded in harbour work:

Highest observed water level	HOWL
Highest astronomical tide	HAT
Mean tidal high-water level	MHW
Mean water level	MW
Mean tidal low-water level	MLW
Lowest astronomical tide (chart datum)	LAT
Lowest observed water level	LOWL

In observing the highest or lowest water level, one should also take into account the changes in atmospheric pressure and the effect of strong winds, either blowing onshore, tending to pile up water against the coast, or blowing water off the coast.

The rise or fall of the water level due to a change in the atmospheric pressure is approximately equal to 0.9 cm rise or fall of the water level for 1 mbar fall or rise in atmospheric pressure. The fall and rise in the atmospheric pressure can, in Norway, give a variation of about maximum ±50 cm. In combination with other effects, such as strong winds, and intensified by geographical constructions this effect can be very important.

The rise of sea level due to the greenhouse effect between years 2000 and 2050 is assumed to be about 0.25–0.30 m.

4.1.3 Water depth

The water depth in the approach channel and the harbour basin, and in the front of and alongside the berth should generally be sufficient for safe manoeuvring. The chart datum for tidal areas should be the lowest astronomical tide and, for rivers, the lowest recorded river level.

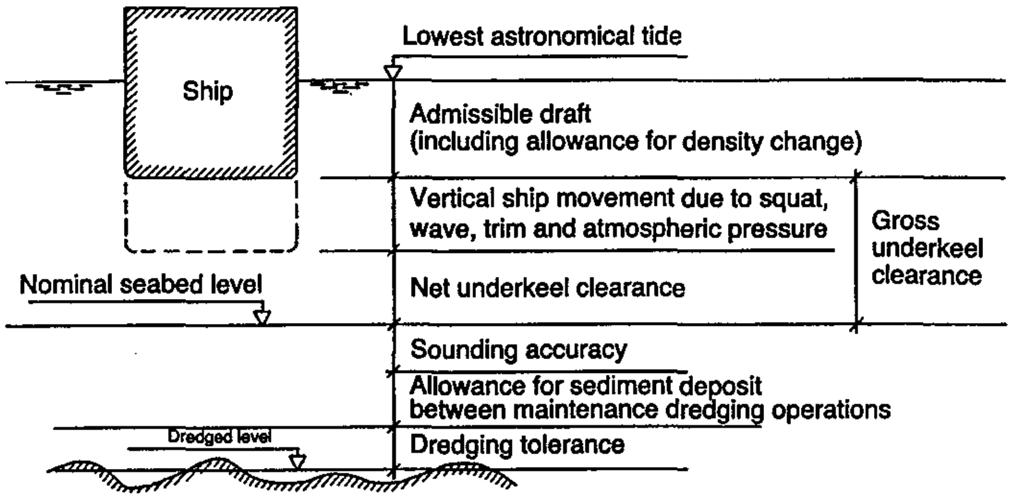


Fig. 4.1. Components of depth

The water depth should be based on the maximum loaded draft of the maximum design ship and can be determined from the following factors:

(a) draft of the maximum loaded ship
(b) tidal variations
(c) movement of the ship due to waves
(d) trim due to loading of the ship
(e) squat
(f) atmospheric pressure
(g) character of bottom
(h) error in dredging
(i) possibility of silting up.

The gross underkeel clearance, as shown in Fig. 4.1 must be designed to allow for waves, trim, squat, atmospheric pressure, etc., in addition to allowing for a safety margin for unevenness of the bottom.

Squat, or the reduction of underkeel clearance, is due to the suction effect, induced by the higher current velocity between the sea bottom and the ship. This causes a reduction in the water level near the ship and the ship therefore sinks bodily in the water. The squat increases with the length of the ship, with the increase in the ship speed, and with reduction in underkeel clearance and narrowness of a channel. In addition, the water depth is also affected by the water density and must be greater in freshwater than in seawater. This can be of importance for river or estuary ports.

Ship movements due to waves can be up to $\frac{2}{3}$ of the significant wave height for smaller ships. VLCC and large ore carriers, due to their huge

Table 4.1. Allowance for water depth in front of a gas jetty for a 136 000 m^3 gas tanker

Depth factors due to	136 000 m^3
Loaded draft (max.)	11.3
Approx. LAT	1.6
Atmospheric pressure	0.4
Movements due to waves	0.5
Trim due to loading	1.0
Safety due to dredging	0.5
Character of bottom, rock	0.5
Water depth below MW	15.8

size, are only susceptible to waves with a period of more than 10 s. Waves with a shorter period will scarcely result in vertical motions for these ships.

Where the bottom is composed of soft materials (sand, etc.) the minimum net underkeel clearance should be 0.5 m and for a rocky bottom 1.0 m. Where the bottom of the seabed consists of silt and or mud, it is usual to define a nautical depth as being from the water surface to the level at which the density of the bottom material is equal to or greater than 1200 kg/m^3 since material layers of a lower density do not significantly impede the passage of a ship.

As an example the allowance for the water depth in front of a gas jetty below land zero level for a 136 000 m^3 gas tanker is shown in Table 4.1.

The water depth must, during construction dredging, be sufficient to avoid both possible errors in dredging and a yearly excessive cost for maintenance dredging due to the possibility of silting up. The water depth must also take into consideration an increase in draft due to the roll and pitch of a ship when moored, as indicated below:

(a) Increase due to roll $= 0.5 \times$ beam $\times \sin\alpha$ where α is the roll angle.
(b) Increase due to pitch $= 0.5 \times L_{OA} \times \sin\beta$ where β is the pitch angle.

As a rough guide, the gross underkeel clearance above the nominal seabed level for the maximum ship using the seaway, should, as a minimum, be the following:

(a) Open sea areas: for high-ship speeds and exposure to strong swells, the clearance should be approximately 30 per cent of the maximum draft.

(b) Exposed channels: exposed to strong swells, the clearance should be approximately 25 per cent of the maximum draft.
(c) Exposed manoeuvring and berthing areas: exposed to swells, the clearance should be approximately 20 per cent of the maximum draft.
(d) Protected manoeuvring and berthing areas: protected from swells, the clearance should be approximately 15 per cent of the maximum draft.

The nominal seabed level is the level above which no obstacles to navigation exist. For good manoeuvring control, the ship requires deeper water depth than the absolute minimum requirement from loading of the ship, tidal variations, trim, etc. In a channel it is desirable to have a ratio of channel depth to maximum draft of the largest ship of 1.3 for ship speeds under 6 knots and 1.5 for higher speeds.

When erosion protection is needed, it is recommended that the bottom protection be placed 0.75 m below the lowest permitted dredged for berths, which are subjected to maintenance dredging. Therefore the level of the bottom protection requires careful study, because the protection installed should not be damaged during maintenance dredging operations. The water depth should therefore take into consideration any maintenance dredging.

At berths where the movement of the largest ships to be accommodated takes place at the higher states of the tide, the underkeel clearance may be achieved by dredging a berth box in front of the berth structure. The berth box should at least have a length of 1.2 times the overall length of the largest ship and a width of 1.5 times the beam of the largest ship that will use the berth.

4.2 Navigation

The total navigation operation, ranging from arrival to departure, can be subdivided into the following operations:

(a) Arrival at the outer harbour basin.
(b) Preparation for berthing, including possible turning of the ship and pre-berthing procedures in the harbour basin.
(c) Berthing including mooring, etc., to the berth structure.
(d) Loading and unloading operations while at berth.
(e) Unberthing from the berth structure.
(f) Departure from the harbour basin.

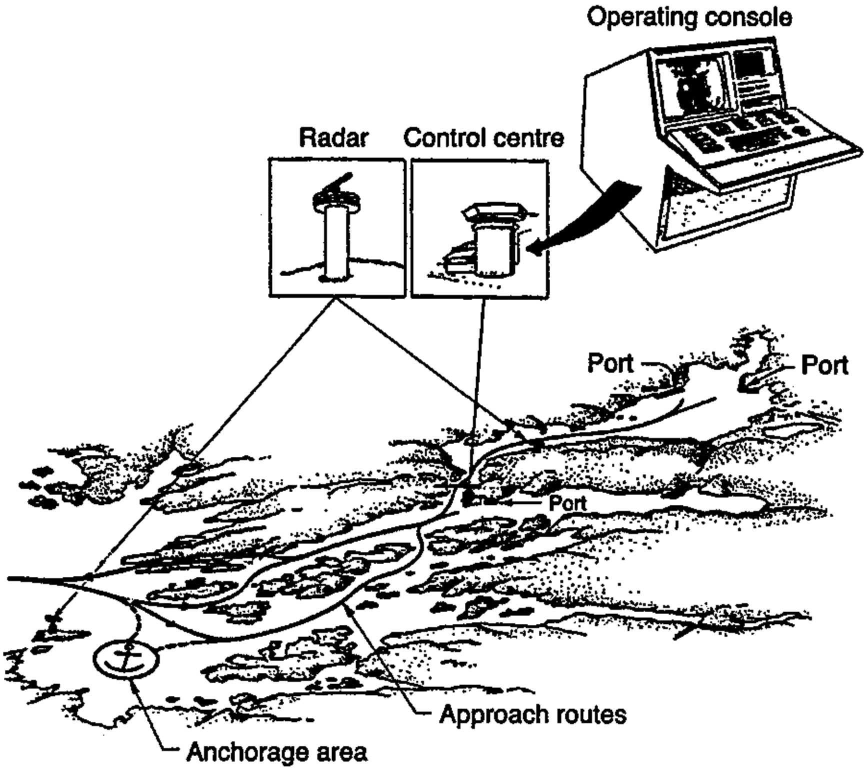


Fig. 4.2. The vessel traffic system (VTS)

Where the ship traffic flow from the open sea to the port is very busy in the approach routes, access channels or harbour area, the total traffic efficiency and safety can be improved by using an **Automatic Identification System** (AIS) and a **Vessel Traffic System** (VTS) such as the Norwegian system Norcontrol I T AS or equivalent. These systems are designed to communicate with each other automatically and exchange critical information about the different ships' course, speed and intended route. In Fig. 4.2 a typical VTS system is illustrated in principle. The radar information is transferred to a traffic-control centre and presented on a bright, high-resolution colour monitor together with detailed chart information.

Ships can automatically be acquired and tracked by the system and shown on the monitor as target symbols with name and vectors indicating course and speed, as shown in Fig. 4.3, and from the traffic-control centre shown in Fig. 4.4 and on the monitor, as shown in Fig. 4.5. Alarm strategies can be set up so that no intervention is required unless an alarm occurs or the operator needs further information. This system can integrate information from several radars and has proven to be an extremely valuable tool in many ports. All data from traffic movements can be stored and used as references for port administration, port authorities, coastguards, and search and rescue services.

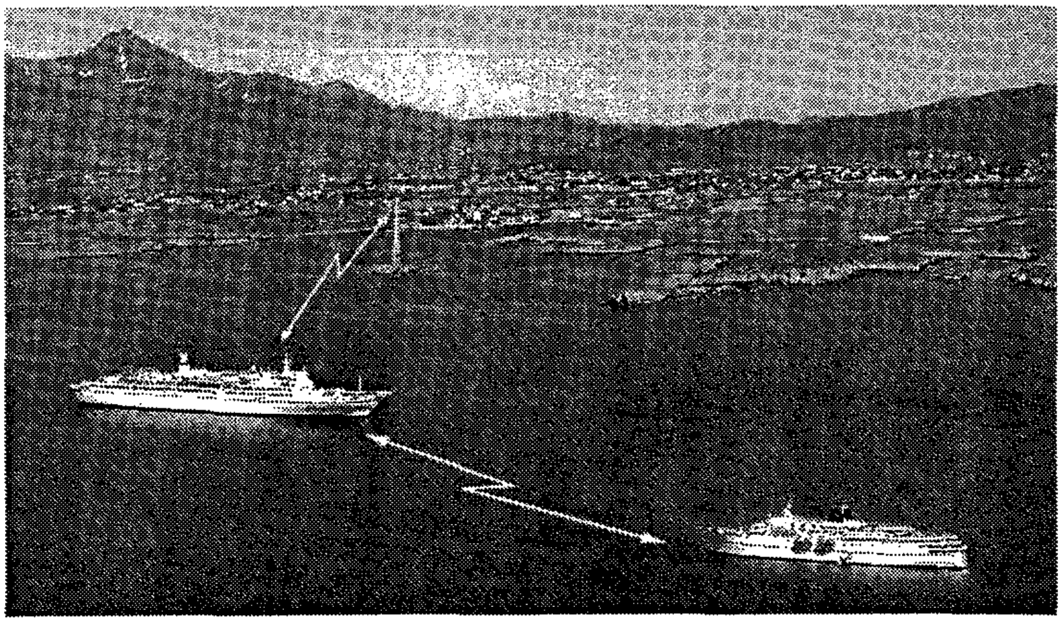


Fig. 4.3. The radar information is transferred to the control centre. Photo courtesy of Norcontrol, Norway

From the traffic-control centre, as shown in Fig. 4.4, the information is presented on a bright, high-resolution colour monitor together with detailed chart information, as shown in Fig. 4.5.

The decision on where the arrival and the berthing operation should start, and where the required number of tugboats should meet and assist

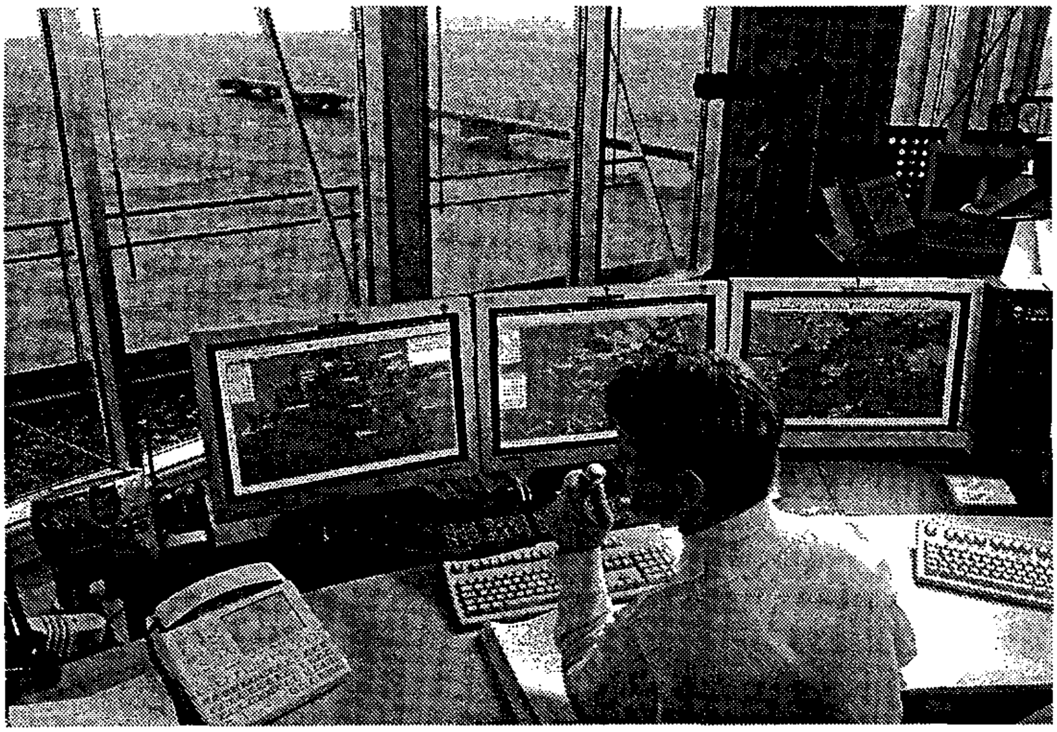


Fig. 4.4. Inside a traffic-control centre. Photo courtesy of Norcontrol, Norway

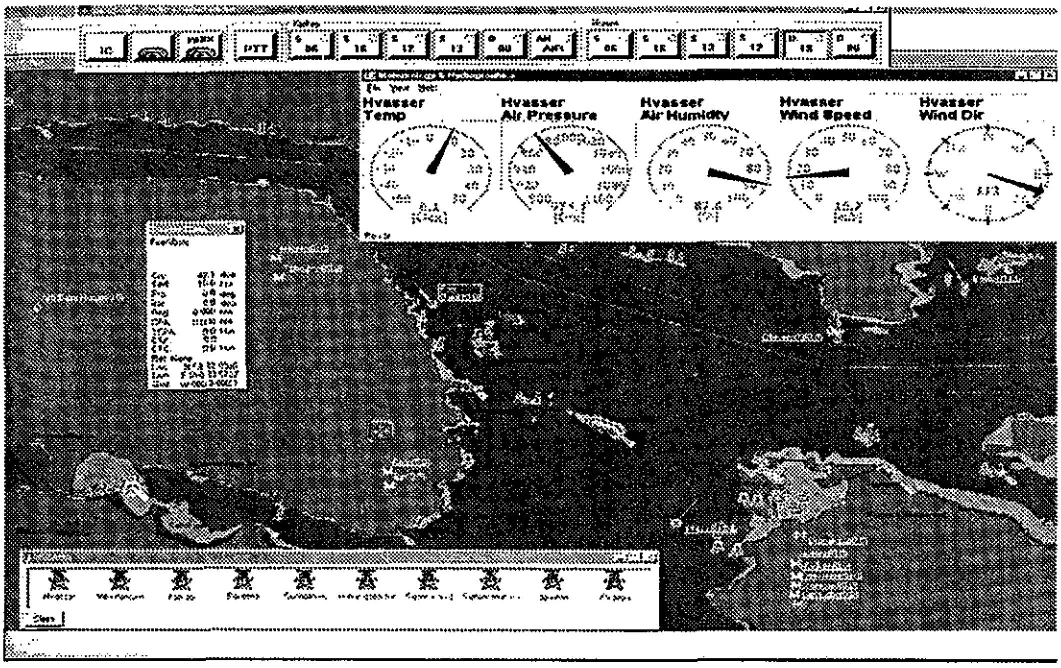


Fig. 4.5. Monitor showing detailed information. Photo courtesy of Norcontrol, Norway

the ship to the berth, must be made by the local pilot or the port captain together with the ship's captain depending on the weather condition, on whether the ship has bow thrusters or not, etc. Pilotage should be made compulsory for all gas and oil tankers and large ships calling at a harbour. The regulations for the harbour will indicate where the pilot should meet a ship.

For the purpose of manoeuvring, all ships should maintain a reasonable ballasted condition. The ships should carry sufficient ballast so that the propeller remains immersed. At some ports, for example oil terminals, the tankers are required to carry a total deadweight of not less than 35 per cent of summer deadweight tonnage, including bunkers, fresh water and stores, on arrival.

Manoeuvring during berthing and unberthing of a ship will generally be done in one of the following ways:

(a) By using only the ship's own engine, rudder, bow thrusters and/or the ship's anchor.
(b) With the assistance of only one or more tugboats.
(c) By using the ship's own anchor, and with the assistance of one or more tugboats.
(d) With the use of berth or land-based winches, and with the assistance of one or more tugboats.

(e) With the use of the mooring buoy, and with the assistance of one or more tugboats.
(f) A combination of two or more of the above-mentioned systems.

Usually nearly all berthing and unberthing operations have been done after any one of case (a), (b) and (c). Case (d), with the use of land-based winches of hauling capacity of about 75 tons can have some technical advantages compared to case (a), (b), (c) or (e). The difference in operation of case (d) is both of a technical and of an administrative nature. From a technical point of view, the efficiency, particularly with an offshore wind, will tend to be higher with winches because the responsiveness to action is much faster than for tugboats. The winch system is also safer because the berthing operation is more punctual and faster than for operations with only tugboats. For berthing of larger tankers the winch system is more economic since it will use fewer tugboats. From an administrative point of view, there may be a shift of responsibility from the tanker's captain to the harbour captain if the mooring ropes are tightened by winches on the berth or land rather than by the tanker's winches. This could cause problems, for instance, with the insurance company if an accident should occur to the berth or the tanker during the berthing operation.

The **nautical chart** in Fig. 4.6 shows the approach route to the terminal at the top of the chart. The nautical chart is from the Norwegian Hydrographic Service map number 18. In Fig. 4.7 the manoeuvring of arriving and departing tankers to the terminal under different prevailing wind directions is shown with the assistance of tugboats or by using the tanker's own anchor together with tugboat assistance.

For emergency evacuation of the tanker in a strong wind without tugboat assistance, for example due to a fire at the loading platform, the tanker may not be able to leave the berth only under its own engine. On the other hand, if the tanker had used its own anchor during the berthing operation, it may be able to leave the berth by using both its own engine and by pulling itself out with the help of the anchor if the wind is less than approximately 7 Beaufort or between 13.9–17.1 m/s.

All important berth facilities, for example oil and gas berths, should be equipped with a monitoring berthing or **docking aid systems** (DAS) comprising the following data:

(a) wind and sea-current sensors detecting speed and direction
(b) wave heights and tide levels
(c) the ship's approach velocity, distance to jetty and angle of approach during all stages of the berthing operation

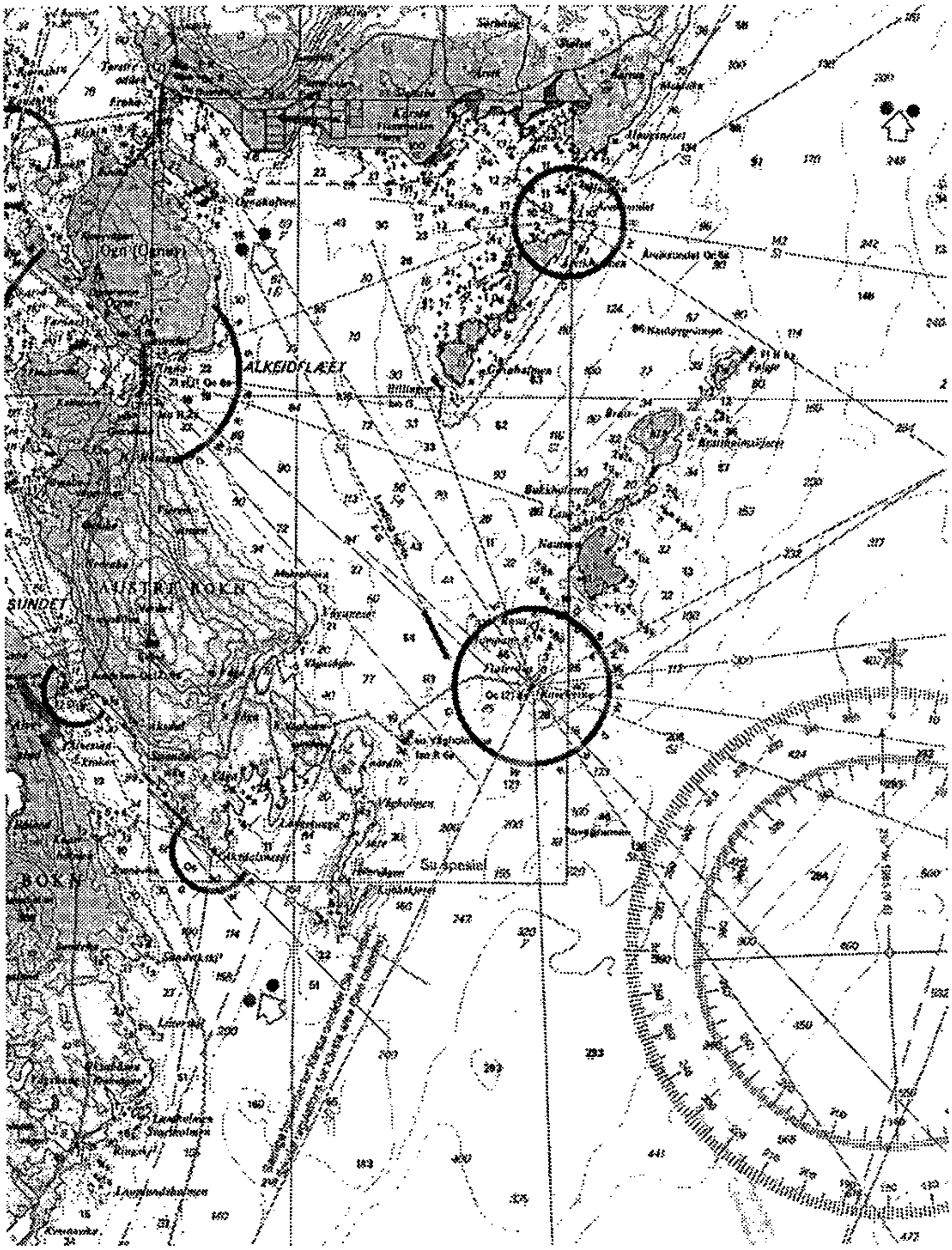


Fig. 4.6. Nautical chart

(d) mooring-line loads to the mooring hooks, pulleys or winches
(e) measurement of ship surge and sway when the oil loading arms are connected.

The purpose of the DAS is, therefore, to measure and track the speed, distance and angle of the approaching ship over the last 300 m until contact is made with the berthing structure and the ship comes

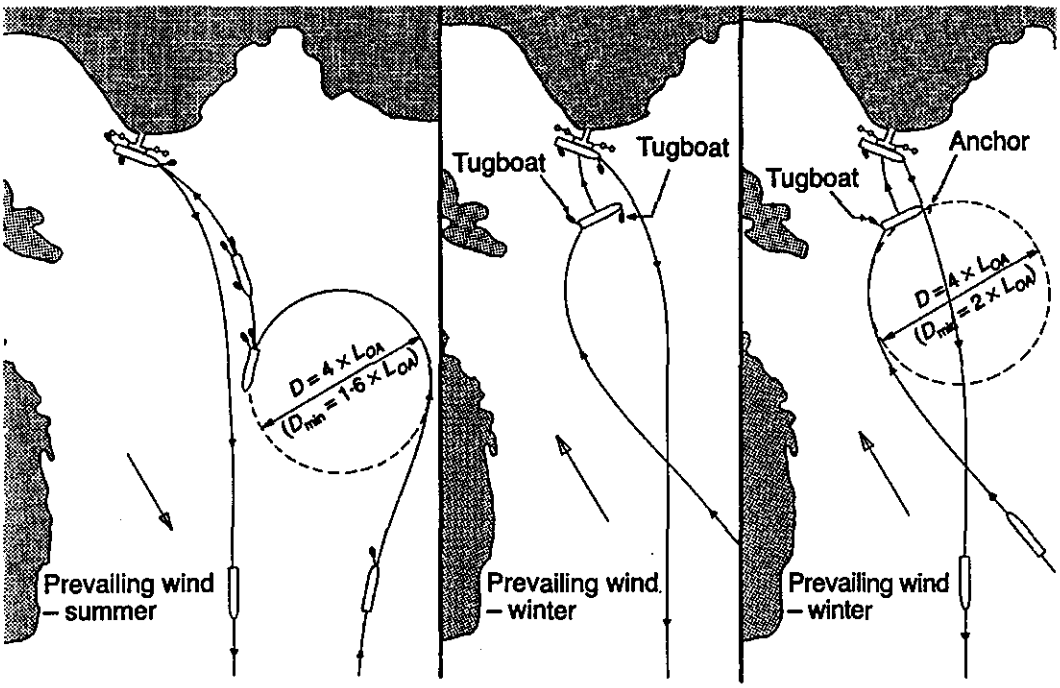


Fig. 4.7. Manoeuvring during arrival and departure

to rest against the breasting fenders. After berthing, the DAS should remain operational to monitor the position of the ship in terms of transverse drift from the fenders face or as compression on the fenders.

Particularly in the petro-chemical and dry-bulk sectors, a large number of terminals now have shore-based berthing aid systems and/or DAS to assist in guiding ships safely to the berth structure. The main berthing aid systems in service are the sonar-in-water and the radar systems, as illustrated in Fig. 4.8 and in more detail in Fig. 4.9. Practical experience has shown that too much reliance should not be placed on the sonar in

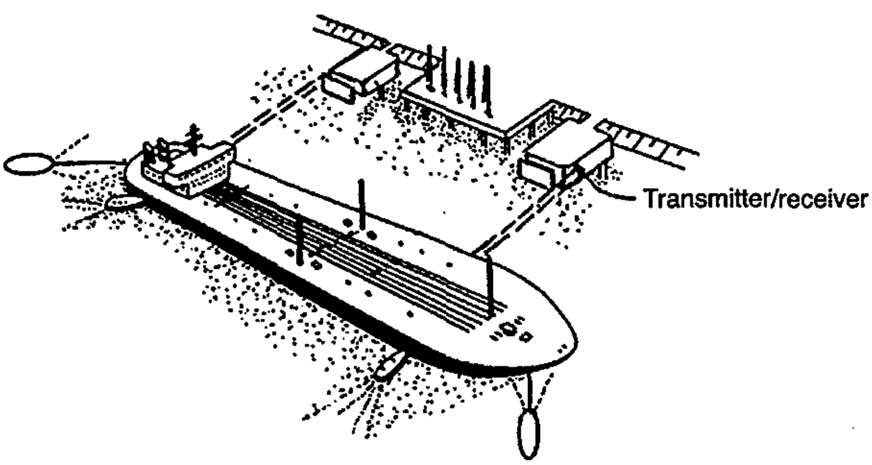


Fig. 4.8. Berthing aid system

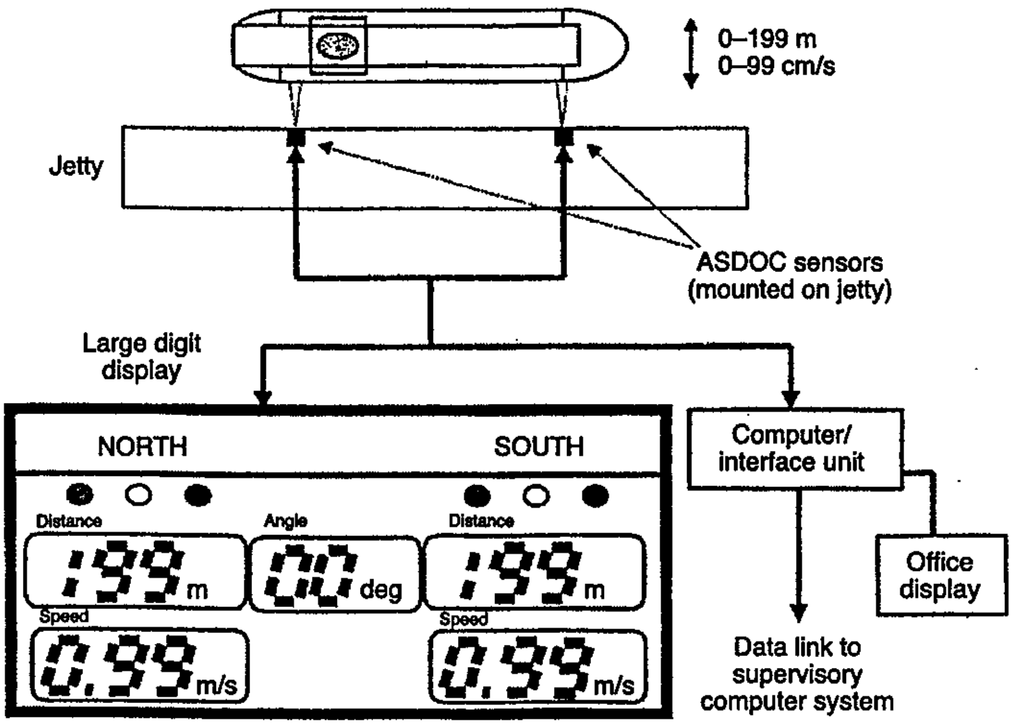


Fig. 4.9. Example of a docking laser system

water system, due to turbulence caused by the ship's propellers, unwanted reflections, maintenance problems in water, etc. The distance between the two laser sensor units should be at least approximately 40 m.

All information should be permanently recorded as part of the berthing and mooring history. To increase safety during the berthing operation, a display board unit should be provided at the berth structure, as shown in Fig. 4.10. The display board should be mounted on a fixed, approximately 2 m high pedestal foundation for measuring both the ship's bow and stern velocity of approach and distance from the berthing line independently. The display board unit should be large enough to allow the ship's captain and the pilot to read information from a distance of approximately 200 m off the berth structure.

All the information from the berthing aid system should be displayed on a visual display unit in the control room showing the berth in plan view, the position of the ship in relation to the berth structure, the actual mooring lines in use and the loads on the mooring lines, the wave heights, the wind speed and directions, and the current speed and directions. A display with the information could be as shown in Fig. 4.11 from Marimatech, Denmark, or an equivalent system.

The berthing aid data usually become available at a distance of approximately 200 m from the berth. All data should be routed to the

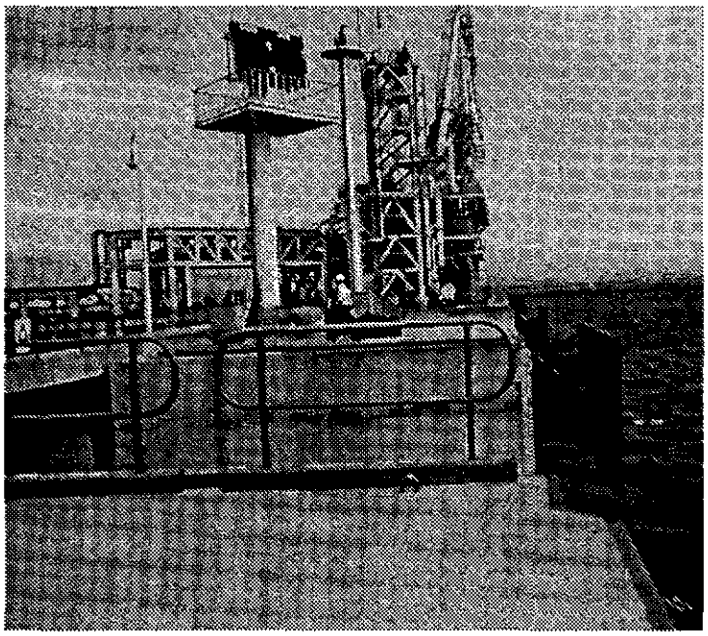


Fig. 4.10. Berth with display board unit

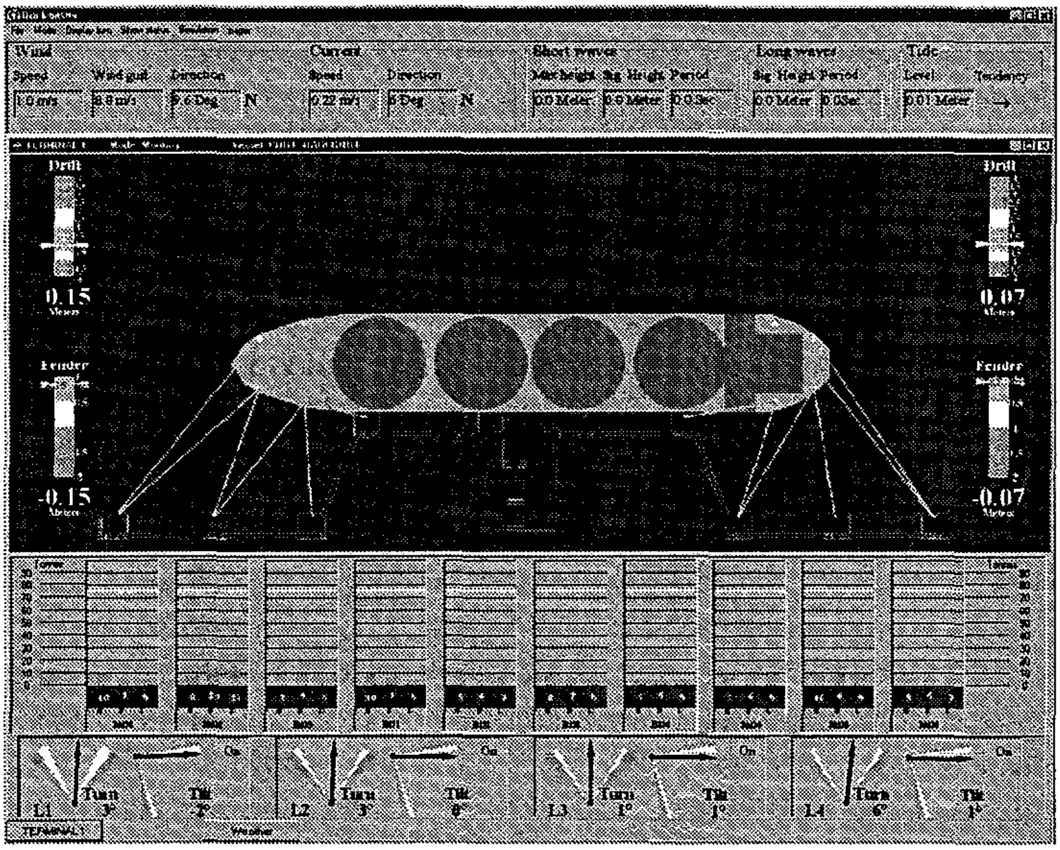


Fig. 4.11. The berthing visual display

terminal control centre where it can be displayed and recorded. The total navigation operation should be watched by radar, and marine and internal radios should also be provided. The mooring operation should be under the command of the tanker captain assisted by the pilot, tugboat captains and terminal mooring supervisor. The terminal mooring supervisor may interfere in the mooring operation if, for some reason, he finds the operation to be hazardous. He may do this by telling the tanker captain to reduce approach speed or, in the worst case, he may deny the tanker mooring permission, stop the berthing operation and instruct the tanker to leave the terminal.

When berthing to an oil or gas berth or terminal, the tanker shall be stopped at a distance of about 100–200 m off the berth, and from this position the tugboats move the tanker transversely at a controlled approach velocity towards the berth. The approach velocity of the ship shall gradually be reduced to about 0.05 m/s in the final phase before the ship hits the fender structures. The maximum acceptable approach velocity should be about $\frac{2}{3}$ of the design approach velocity for the berth structures. When closing up to the berth, the mooring boats or launches should bring the fore and aft breast mooring lines from the tanker to the mooring points at the berth structure. The tanker's spring mooring lines should then be moored to the berth structure. The tanker's position can now be adjusted by using the fore, aft breast and spring mooring lines, together with the tugboats. The berthing angle between the ship and the berth line during the final stage of the berthing operation should not be more than 3°–5°.

In addition to the electronic monitoring systems installed at the berth for measuring the wind, wave, current and tide, it is advisable to install an additional visual wind indicator similar to a windbag of the type that normally is installed at airports and airfields. At a mast on top of the control building, which should be situated in a non-hazardous area, one should measure the wind, temperature, humidity, air pressure and visibility.

The shore-based berthing aid system can play a major role in reducing damage to the berth and fender structure and/or the ship. It is undeniable that millions of pounds worth of damage is caused annually to berth structures and/or ships around the world through ships approaching too fast, at the wrong angle, etc. From terminals with berthing aid systems it has also been found that the berthing time over the last 20 m between the ship and the berth structure had been reduced to 30–40 per cent.

4.3 Tugboat assistance

The efficient and safe manoeuvres of large ships in confined waters are largely determined by the degree of controllability of the ship's own rudder, propellers and thrusters and by the assistance of necessary tugboats due to weather conditions. The ship's captain should, before entering restricted areas of a port, also ensure that anchors are ready for letting go prior to entering the pilot operating areas.

From a safety point of view, tugboat assistance should always be used during berthing and unberthing operations of oil and gas tankers. Particularly during the berthing operation one should always use tugboats, so that the tanker can approach the berth structure parallel to the fender face of the berth. The tugboats will guarantee a simpler berthing and unberthing procedure, and also represent an important safety factor if the ship develops engine or steering trouble. The tugboats may also be used to assist other traffic in the lead. It is also strongly recommended that, under normal circumstances, oil and gas tankers should not leave the berth without the assistance of tugboats. However, from an emergency evacuation point of view it is recommended that they are moored in such a way that unberthing operations will be as easy as possible.

The term 'tugboat escort' refers to the stationing of tugboats in the proximity of the ship as the ship is manoeuvring into port to provide immediate assistance if a steering and or propulsion failure should develop. The term 'tugboat assist' refers to the situation when the tugboat is applying forces to assist the ship in making turns, reducing speed, berthing, etc.

A ship will generally start to lose steerage if its speed, under its own engine, is less than about 3–4 knots depending on the type of ship and on whether the ship has bow thrusters. Therefore, when berthing an oil tanker, the tugboats should meet the tanker outside the restricted oil harbour area and before the speed of the tanker is less than about 3–4 knots, and connect up the tow lines, at least one fore and one aft.

The requirements of the tugboat fleet shall cover the following services:

(a) Provide necessary assistance during the berthing and unberthing operations to counteract the wind, wave and/or current forces.
(b) Enable the ship to turn in a confined area.
(c) Act as a restraining or anchoring force on a ship moving towards the berth structure.
(d) Act as a stand-by ship when a gas or oil tanker is moored.

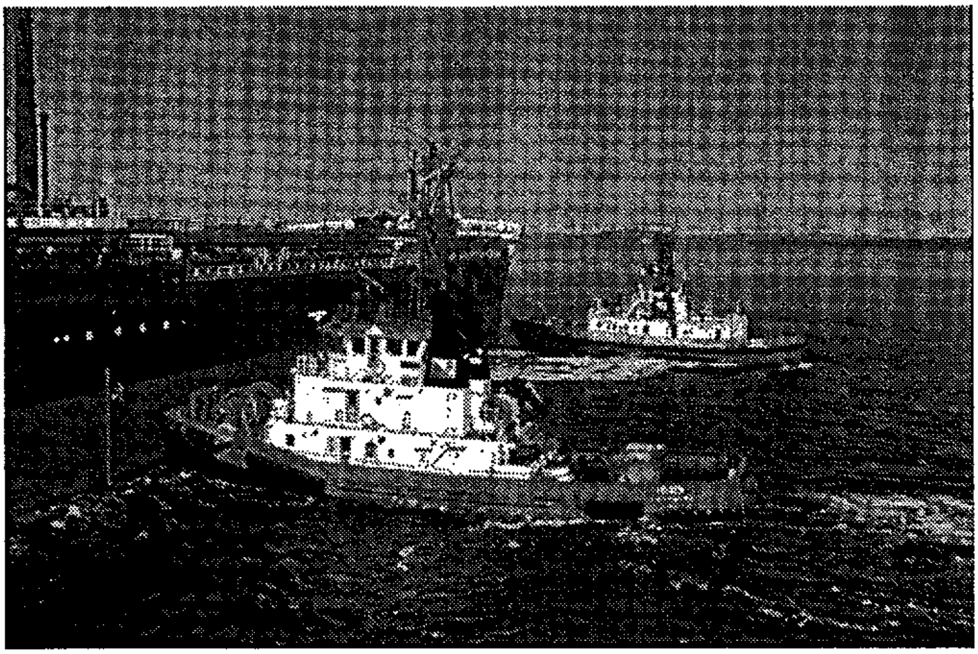


Fig. 4.12. Tugboats assisting during the berthing operation. Photo courtesy of Buksér og Berging AS, Norway

(e) Carry out emergency, fire fighting and antipollution operations. For a ship in an emergency situation, for example due to breakdown of propulsion machinery, steering gear, etc., the tugboat must be able to assist directly.

The steering and manoeuvring capacity of a ship is dependent on the following:

(a) the environmental condition (wind, waves and current)
(b) the mechanical equipment of the ship itself (the main propeller and rudder, bow thrusters, etc., and the available tugboat assistance).

Generally, depending on the wind and/or wave direction, the pilot and the ship's captain will normally decide which side shall be alongside the berth. However, the ship shall, if possible, be moored with the head out in event of an emergency leaving. The number of tugboats assisting in each berthing and unberthing operation will, as shown in Fig. 4.12 from Buksér og Berging AS, Norway, depend on the weather conditions at that time.

Tugboats can generally be divided into two groups according to their size and power:

(a) Harbour tugboats which operate mainly in sheltered waters. Their horsepower varies roughly between 2000 HP and 7000 HP, and their sizes between about 15–30 m length.

(b) Offshore tugboats which can operate in exposed waters. Their horsepower varies roughly between 10 000 HP to 20 000 HP or more, and their sizes between about 30–60 m length.

The necessary tugboat HP is approximately 10–12 times the actual tugboat bollard pull in kN. One HP is equivalent to approximately 0.75 watts.

The number of tugboats needed for handling the different types of ships is affected by the size and type of ship, the approach route, the exposure and type of the berth structure, the environmental conditions, etc., and on the bollard pull that each tugboat can mobilize.

Therefore the necessary tugboat capacity in effective bollard pull should be sufficient to overcome the maximum wind, waves and/or current forces generated on the largest ship using a port, under the maximum wind, waves and/or current permitted for harbour manoeuvring and with the ship's main engines and bow thrusters out of action.

In the evaluation of the total number of tugboats required for manoeuvring of a ship under the berthing and unberthing operations, the following must been assumed:

(a) The ship is not equipped with bow thrusters.
(b) The forces acting on the ship can be due to wind (F_{wind}), wave (F_{wave}) and current ($F_{current}$). For the wind forces a 'gust factor' (F_g) of about 1.2 should be applied.
(c) The force required to move a ship against the wind and current is at least generally assumed to be approximately 30 per cent higher than the forces necessary to hold the ship against the forces due to wind and current.

Due to possible uneven bollard pull when several tugboats are used, and inaccuracy in the method of calculating the bollard pull required to control the ship, it is usually recommended that a tugboat bollard pull factor S_f of between 1.2 and 1.5 is used depending on the general weather conditions.

The **total required effective tugboat bollard pull B_P** needed to control a ship due to environmental forces can be calculated approximately from the following formula:

The effective tugboat bollard pull B_P that is needed

$$= S_f \times [(F_{wind} \times F_g) + F_{wave} + F_{current}]$$

An evaluation of the necessary tugboat bollard pull should at least be based on the design specification from British Standard BS 6349, Part 1 (formulas for wind and current forces), or OCIMF, the Oil Companies International Marine Forum's Mooring Equipment Guide for the Safe Mooring of Large Ships at Piers and Sea Islands (formulas for wind and current forces), SIGTTO, The Society of International Gas Tanker and Terminal Operators Ltd (formulas for wind) and/or in the Spanish Standard ROM 0.2-90 (formulas for wind, wave and current forces) if a model of the specific ship has not been tested in a laboratory.

The Society of International Gas Tanker and Terminal Operators Ltd, recommend that there must always be sufficient tugboat assistance to control the gas tanker in the maximum permitted operating condition and this should be specified assuming the tanker's engines are not available. It is recommended to have three, or preferably four, tugboats to assist the tanker, and that they should be able to exert approximately half of the total tugboat power at each end of the gas tanker.

It is recommended that one always have an **operational safety factor** $S_O \geq 1.1-1.25$ due to the actual total available tugboat capacity including any bow thrusters equal T_C during berthing and/or unberthing divided by the total needed effective tugboat bollard pull B_P.

Most of the oil terminals, and especially the gas terminals, around the world require, due to safety reasons, that one always use at least four tugboats during berthing to and three during unberthing from the berth.

The presence of an oil or gas tanker at a terminal will involve a certain risk to the surroundings. For safety reasons, one should therefore always have at least one tugboat on stand-by in case of possible changes in the weather situation and/or need for emergency departure due to fire, etc.

Example of the total forces acting on a ship

The approximate wind and current area dimensions in m^2 for a 137 000 m^3 spherical LNG gas tanker in ballasted condition, length $L_{OA} = 290$ m, length $L_{BP} = 270$ m, width = 48.1 m and ballasted draft = 7.5 m is used in the example of the total forces acting normal to the tanker in Table 4.2.

The total force acting normal or crosswise to the 137 000 m^3 spherical LNG tanker in ballast condition in open sea (deep water), will, after the OCIMF/SIGTTO recommendation (wind forces), the British Standard (current forces) and the Spanish Standards (wave forces), need a total effective bollard pull B_P (with a wind (10-min mean), a gust factor of 1.2 and an uneven tugboat bollard pull factor $S_f = 1.3$) as follows:

Table 4.2. Total forces acting normal to the tanker

Area of spherical LNG tankers	137 000 m^3
Wind laterally projected area:	
Ballast	8 400
Wind front area:	
Ballast	2 100
Current laterally projected area:	
Ballast	2 100
Current front area:	
Ballast	360
Total current surface area:	
Ballast	13 000

(a) For a wind speed of 13 m/s, a wave height of 1.0 m and a deep-water current speed of 0.5 m/s, the total bollard pull

$$B_P = S_f \times [(F_{wind} \times F_g) + F_{wave} + F_{current}]$$
$$= 1.3(1484 + 178 + 265) = 2505\,\text{kN}$$

(b) For a wind speed of 10 m/s, a wave height of 0.0 m and a deep-water current speed of 0.0 m/s, the total bollard pull

$$B_P = S_f \times [(F_{wind} \times F_g) + F_{wave} + F_{current}]$$
$$= 1.3(878 + 0 + 0) = 1141\,\text{kN}$$

From this example, it should be recommended that at least four tugboats are stationed at the terminal for handling 137 000 m^3 tankers at approximately 13 m/s, each with approximately 6000 HP and a minimum pulling capacity of 70 ton, which would give a total tugboat pulling capacity T_C equal to 2880 kN. The operational safety factor S_O would then be 1.15.

Figure 4.13 shows the necessary tugboat bollard pull based on British Standard BS 6349 (see Chapter 2 for wind and current forces), required to move different sizes of oil tankers in deep water with winds of 10 m/s and 15 m/s and a gust factor of 1.2 and a current of 0.5 m/s, acting crosswise to the oil tanker. For wind and current areas, see Chapter 20 for ship dimensions. A tugboat bollard pull factor of 1.4 has been used. From the figure it can generally be seen that for a ship in ballasted condition the wind forces are the dominating forces acting on the ship, while in loaded condition the current forces are the important forces.

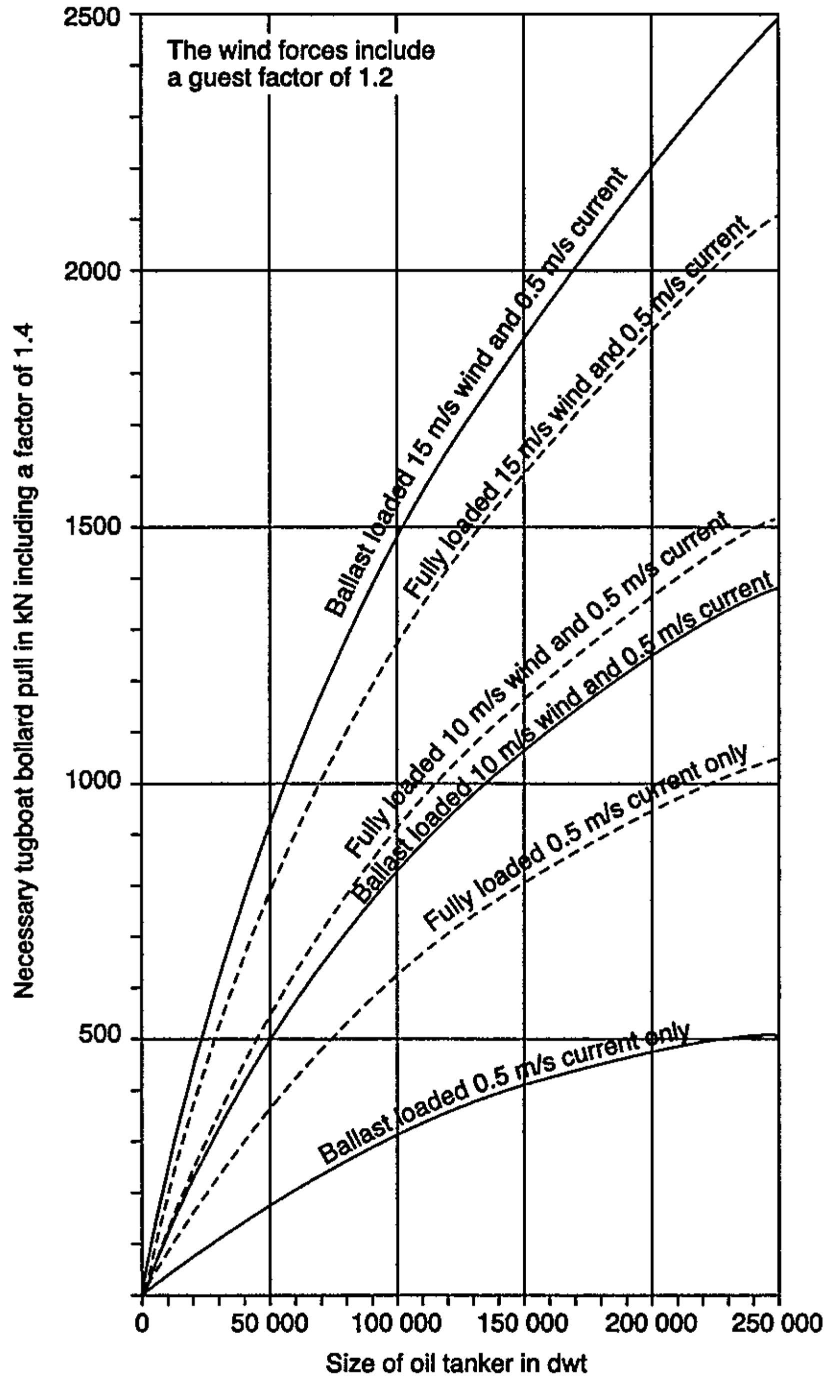


Fig. 4.13. Necessary tugboat bollard pull for crosswise wind and current acting on an oil tanker at deep water

From Fig. 4.13 it can also be seen that the necessary tugboat bollard pull in kN is roughly equal to about 0.5–2.5 per cent of the tanker's dwt including the reserve capacity.

The necessary required tugboat pull in kN due to a crosswise current acting on a ship as a function of the ratio of water depth to ship draft, based on the general standard current formula shown in Chapter 2

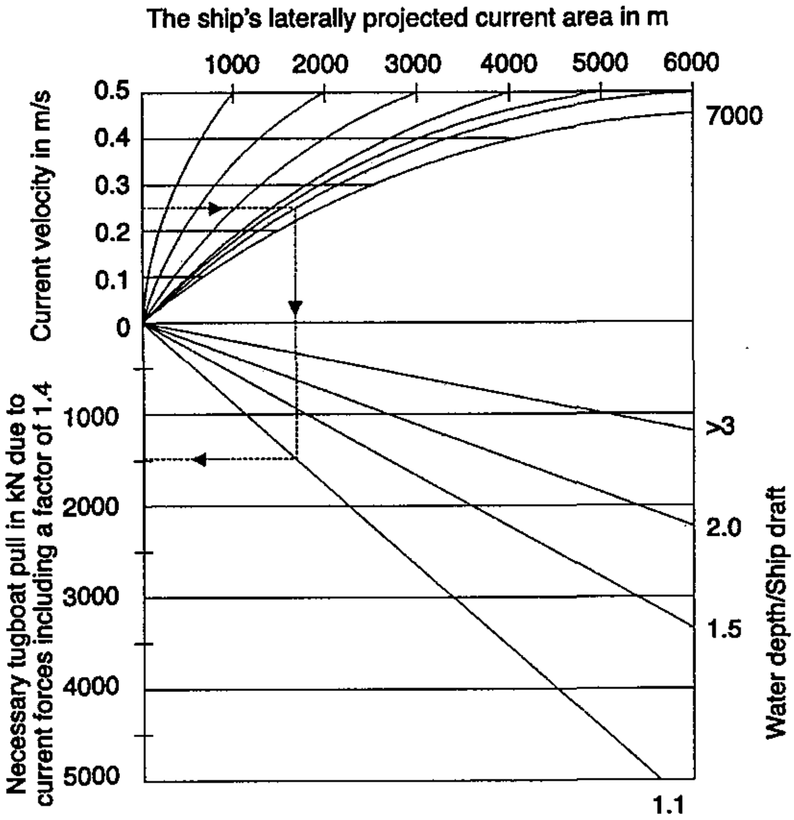


Fig. 4.14. Necessary tugboat pull due to a crosswise current acting on a ship as a function of water to ship draft

for current forces, is shown in Fig. 4.14. A tugboat factor of 1.4 has been used.

As illustration Fig. 4.15 shows the number of tugboats usually required to assist an oil tanker of between 100 000–200 000 dwt and/whilst

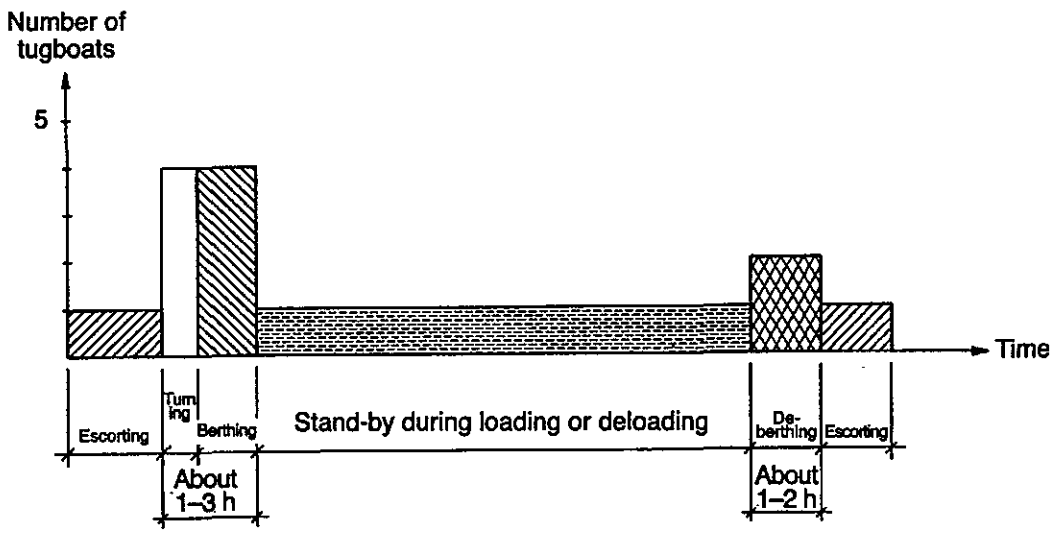


Fig. 4.15. Required number of tugboats during different port operation

escorting the ship to the harbour basin, turning, berthing, stand-by during loading or unloading, unberthing and escorting out of the harbour basin in good weather conditions (below Beaufort 5) and with nearly no current, is shown in principle. The time or duration of each of these activities will depend on the sizes of tugboats, type and size of tanker, layout of the harbour and loading or unloading capacity of the piping system.

4.4 Wind and wave restrictions

Generally the terminal operator and the port captain should have a good understanding of the mooring principle for all the ships using the terminal and the different berths. This includes the fundamental principle of the design of a mooring system and the loads likely to be expected in the mooring system under different conditions of wind, wave and current, and they should have a clear understanding of the operating limits of the various types of ships and mooring arrangements and systems which may be used at the berth.

The following limits for wind velocities, as indicated in this chapter, are commonly used in the evaluation of the mooring system, horizontal forces, etc., during berthing and unberthing operations for cruise ships, and for oil and gas tankers shown in Figs 4.16 and 4.17.

For port and ship operations it is generally accepted that a wind gust duration shorter than about 1 min will be of secondary importance.

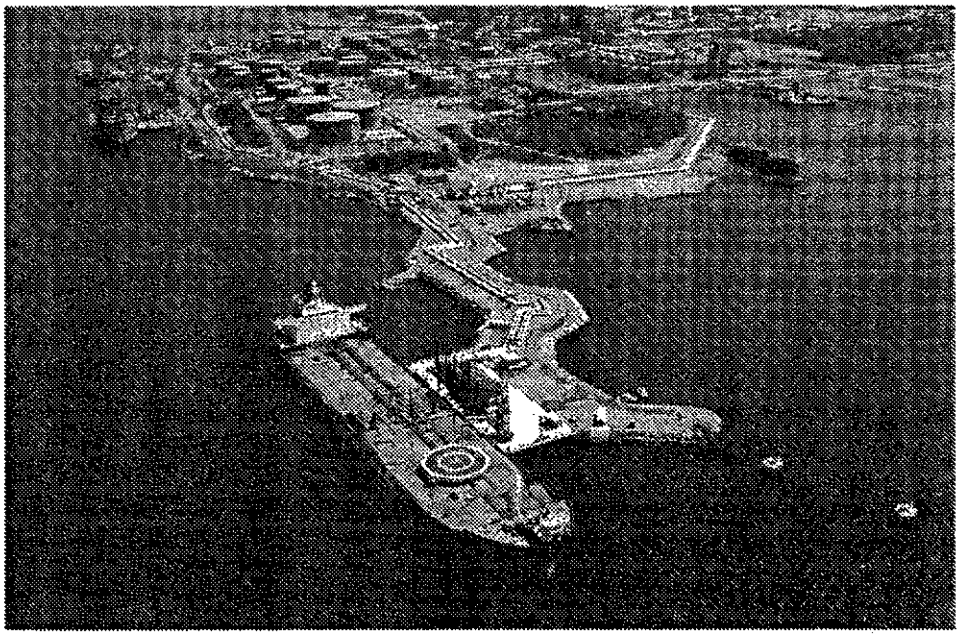


Fig. 4.16. Oil tankers at Statoil Mongstad Oil Refinery. Photo courtesy of Øyvind Hagen, Statoil Mongstad, Norway

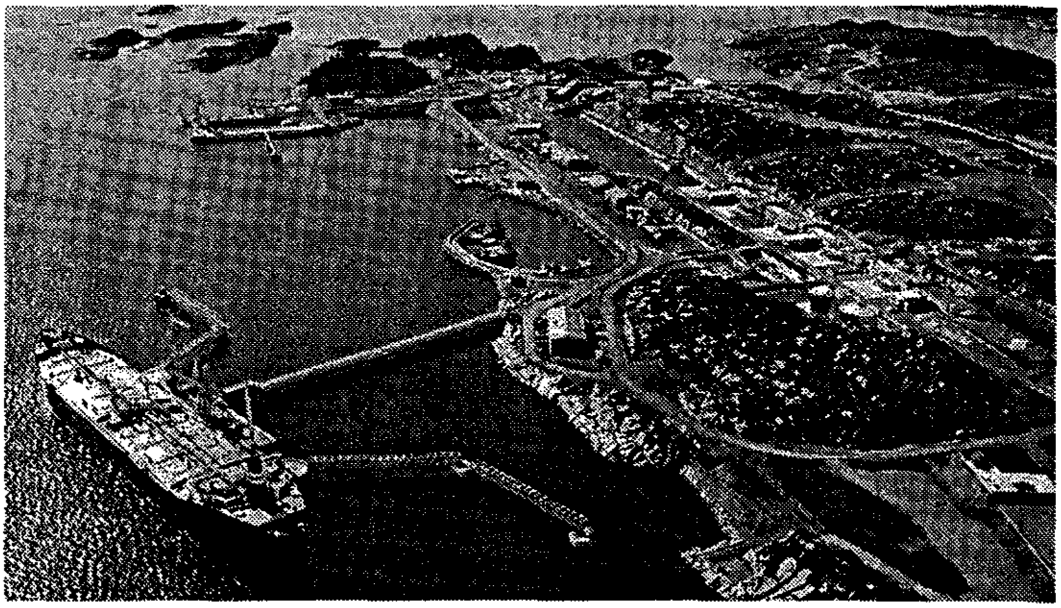


Fig. 4.17. Norsk Hydro, Sture Crude Oil Terminal, Norway. Photo courtesy of Norsk Hydro, Norway

Nevertheless, at some gas and oil terminals around the world it has been observed that wind durations down to 20–30 s may affect the tankers in ballast condition.

Therefore, the design wind speed for a moored tanker should be the mean wind speed corresponding to the shortest gust which will affect the tanker at any time, having a return period of at least 100 years and taking account of the height of the tanker and the wind speed/height gradient. In the case of a moored tanker, the gust duration must also be sufficient for the mooring line or fender strains to develop, taking account of the inertia of the tanker.

For oil and gas berths it is recommended that there is instrumentation for the continuous measuring of the wind velocities in the near vicinity, as indicated in Section 4.2. Guidelines for operating wind limits and mooring arrangements have been developed for all larger terminals to be used by the operators.

A critical relationship for the manoeuvrability of ships with very high superstructures, such as oil tankers in ballast, containerships and car carriers, appears to be the ratio of wind speed to ship speed. At ratios of wind speed to ship speed of about 6–7, great difficulties can be expected in controlling these lightly loaded ships or ships with high windage areas. On the other hand, at ratios of about 10, control of even fully loaded ships, such as large fully loaded tankers, will most likely be impossible.

The wind force on a ship will vary with the exposed area of the ship. A beam wind will strike the entire exposed side area of the ship, compared with the relatively small exposed area for the head wind. For a given wind velocity on, for example, a large tanker, the maximum transverse or crosswise wind force is about three to five times as large as the maximum longitudinal wind force.

The values below are based on 10-min average wind velocities (the Beaufort wind scale). The figures will also depend upon the wave and current situation at the berth.

It is very important that all ports and terminals should give documentation that they can handle ships, taking account of the operational wind speeds, with the tugboat fleet stationed at the terminal.

As a very rough guideline for operation of a berth or terminal, based on experiences around the world, the following **operational wind velocities** are suggested as limits for cruise ships, ferries and for oil and gas tankers in ballast condition during the following operations.

Cruise terminals

One should try to minimize cruise ship motions in winds up to 18 m/s or 35 knots to ensure that the gangways remain operationally safe for arriving and departing passengers and cargo transfer.

Ferry terminals

Due to the large differences between the different ferries due to the hull form and windage area, propulsion system, rudder arrangements etc., it is difficult to recommend wind limits during berthing and unberthing. For larger ferries the wind limits for berthing may vary between 15–30 m/s and for unberthing between 12–35 m/s depending on the wind directions.

It must be remembered that the ferry and cruise terminals are nearly always in locations sheltered from the effects of wave and current.

Oil terminals

For oil tankers the following wind limits could be recommended:

(a) Approximately 10 m/s or 20 knots during berthing of tankers with a laterally projected wind area of more than about 5000 m^2. This is for an oil tanker of more than about 150 000 dwt.
(b) Approximately 15 m/s or 30 knots during berthing of tankers with a laterally projected wind area of less than about 3000 m^2. This is for an oil tanker of less than 60 000 dwt.

(c) Approximately 20 m/s or 40 knots during loading and unloading operations.
(d) The loading arms should be disconnected at approximately 23 m/s or 45 knots, for example, due to movements of the tanker due to wind and waves, or to the manufacturer's specification.
(e) Maximum 26 m/s or 50 knots at berth, but ballasted to reduce the wind area of the ship and with emergency mooring wires. In this case at least one tugboat should be on standby to assist any tanker alongside the berth if required.
(f) At a wind forecast of more than 26 m/s or 50 knots wind velocity, the tanker shall, if possible, normally leave the berth for open sea.

At some large new oil terminals, due to environmental and safety requirements, the acceptable wind limit during berthing is a maximum of 10 m/s for all tanker sizes. The operational limit for the loading arms during loading and unloading is 15 m/s, and the loading arms are disconnected at a wind speed of 20 m/s.

At oil terminals where **double mooring** are used, where one smaller tanker is moored outside a larger tanker at a jetty, the operation will normally stop and disconnection between the tankers will take place if the wind exceeds 16 m/s. If prognoses for the wind will exceed 20 m/s, the smaller tanker will depart for open sea from the terminal.

In Fig. 4.18 the rough operational guidelines for oil tankers, as described above, are shown as a diagram.

Figure 4.19 shows an example of the type of information that the OCIMF recommend should be valuable to the berth operator for mooring 250 000 dwt tankers. The mooring arrangement with 18 nylon lines shows the maximum freeboard for this type of tanker (or minimum draft) and that the loading arms should be disconnected at a wind speed of approximately 30 knots peak gust. As the draft increases the permissible wind speed can increase to 50 knots peak gust. The ship motions, in this case, can govern the loading arms' limits.

For comparison it is generally recommended that equipment such as heavy-lifting equipment for cargo and containers, loading towers, etc. will not operate in a wind stronger than about 20 m/s, and that the wind can blow horizontally in all directions. Therefore, for wind above a certain wind speed level, the operation of the cranes, etc. may be impossible. This need not necessarily affect the total loading and unloading operations of the ship, if these operations can be done by trucks, etc., and provided that the wind has not given the ship itself unacceptable ship movements.

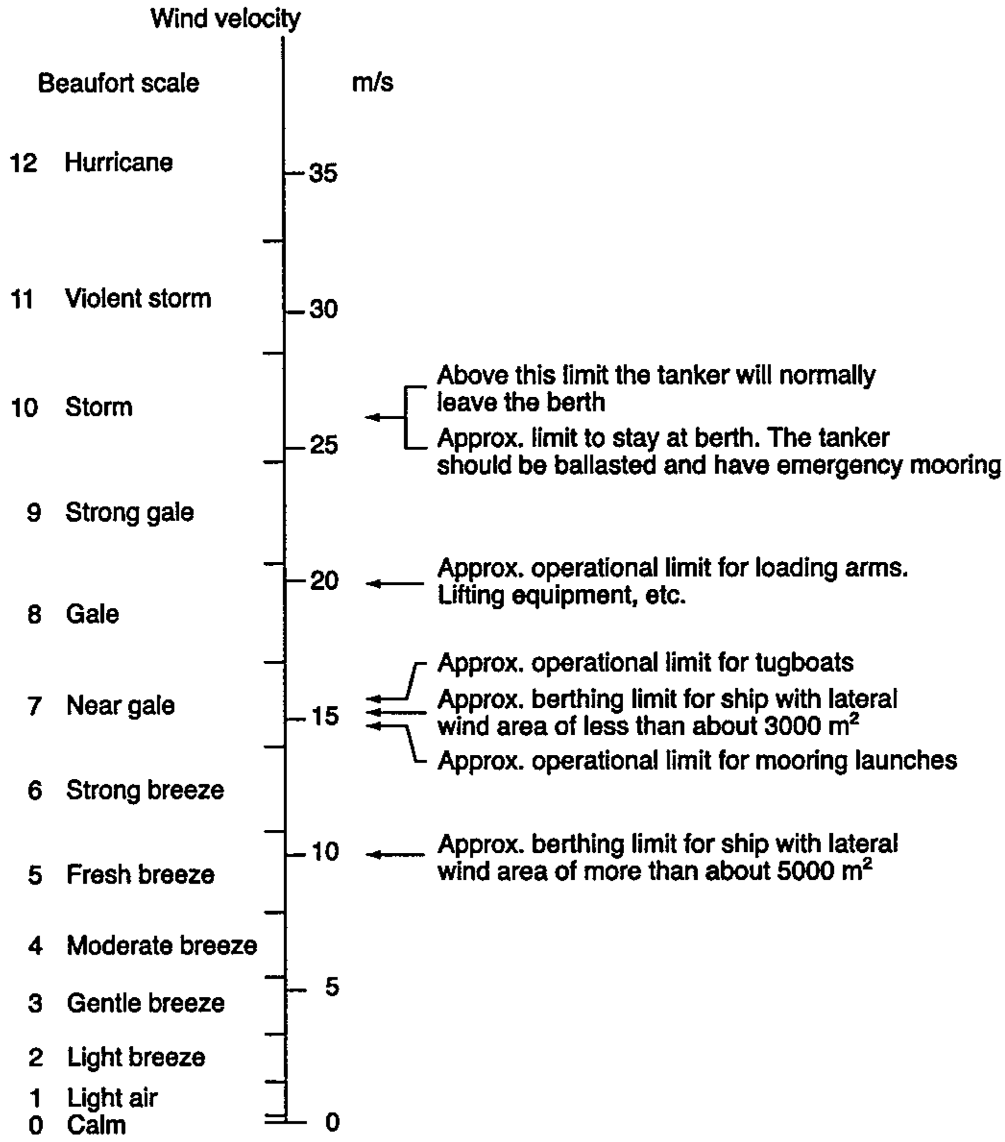


Fig. 4.18. Approximate operational guidelines for tankers

Gas terminals

Due to the higher risk when taking gas tankers in ballasted condition to a berth compared to oil tankers, the acceptable wind velocity limits could be as shown below:

(a) Approximately 10 m/s or 20 knots during berthing of tankers with a laterally projected wind area of more than about 5000 m^2. This is for an LPG gas tanker of more than about 80 000 m^3. A 137 000 m^3 LNG spherical tanker has a lateral wind area in ballast condition of approximately 8400 m^2.

(b) Approximately 12 m/s or 24 knots during berthing of tankers with a laterally projected wind area of less than about 4000 m^2. This is for a gas tanker of less than 70 000 dwt.

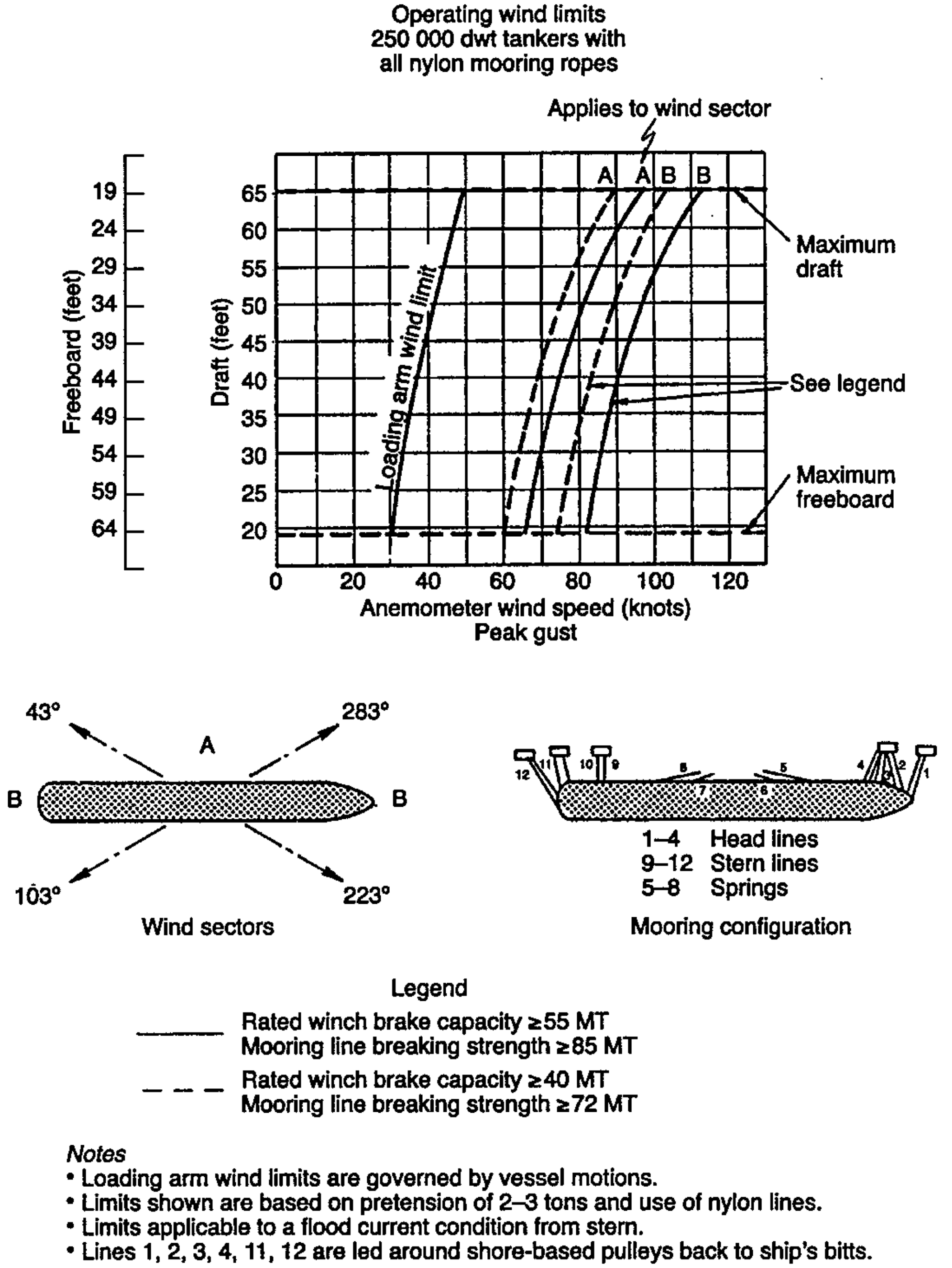


Fig. 4.19. Example of operating wind limits of a 250 000 dwt tanker moored with nylon lines

(c) Approximately 14 m/s or 30 knots during berthing of tankers with a laterally projected wind area of less than about 3000 m^2. This is for a gas tanker of less than 70 000 dwt.

(d) Approximately 17 m/s or 35 knots during loading and unloading operations.

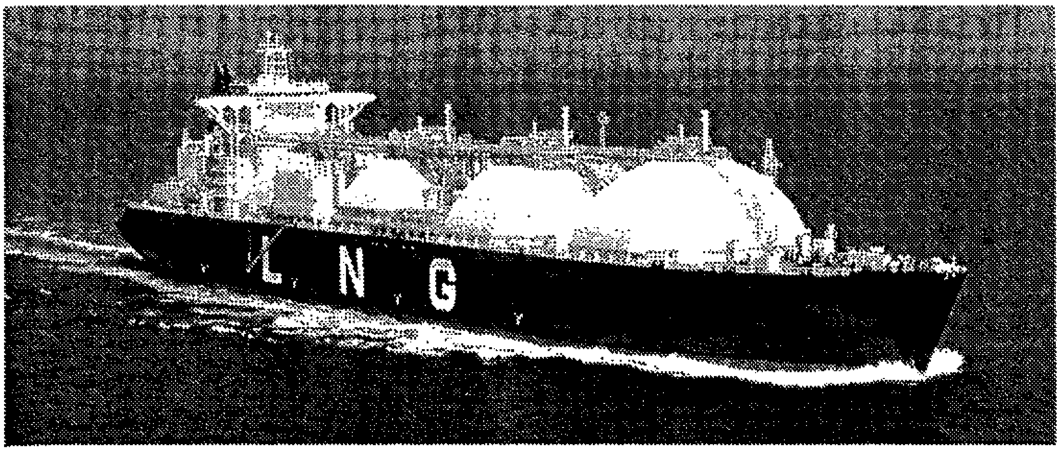


Fig. 4.20. A spherical LNG gas tanker. Photo courtesy of Høegh, Norway

(e) The loading arms should be disconnected at approximately 20 m/s or 40 knots, for example, due to movements of the tanker as a result of wind and waves, or to the manufacturer's specification.
(f) Maximum 24 m/s or 48 knots at berth, but ballasted to reduce the wind area of the ship and with emergency mooring wires. In this case at least one tugboat should be on stand-by to assist any tanker alongside the berth if required.
(g) At more than 24 m/s or 48 knots wind velocity, the tanker shall, if possible, leave the berth for open sea.

It must also be remembered that gas tankers usually have greater free-board than oil tankers, as shown in Fig. 4.20 of a LNG spherical gas tanker, and hence will be more affected by the wind. The maximum acceptable wind velocities are also usually a little higher for smaller ships.

For LNG tankers, the membrane tank LNG carriers have smaller dimensions than the spherical tank LNG carriers of similar capacities, which is of great importance in the determination of wind force effects. At some terminals for approximately 138 000 m^3 LNG, the wind speed limit for berthing is 14 m/s for flat deck or membrane tankers and 12 m/s for spherical tankers.

Tugboats and mooring boats

For tugboats it shall be recognized that, due to wind generated or short periodic waves, the tugboats will have operational limits. With significant wave heights of more than 1.0–1.5 m for ordinary tugboats and approximately 1.5 m for tractor tugboats, tugboats start to lose efficiency in controlling ships.

For modern mooring boats or launches, a wind speed of about 12–15 m/s or a significant wave height of 1.0–1.3 m must be taken as guideline limits for safe operation. If these limits are exceeded, the mooring boats will experience difficulty in delivering the mooring lines from the ship to the mooring points at the berth.

4.5 Ship movements

It is very difficult to predict the acceptable movements for the different types of ships. It can be done with varying degrees of accuracy and reliability by mathematical models and analytical methods, but the most reliable method of predicting a ship's response under wave action is to build and test in a physical model. In this section some recommendations to acceptable movements are given.

Under the impact of current, waves and wind gusts a moored ship is in continuous movement. The magnitude of the movement varies over a wide range, and depends on the magnitude and direction of the waves and wind. Even the best mooring systems will not be able to stop the ship from moving due to waves and wind. The six main components of ship movements are shown in Fig. 4.21.

Usually all movement of a ship will be a combination of more than one of the six movements shown in Fig. 4.21. These six movements can strictly speaking be subdivided into two main types of movements or oscillations:

(a) Movements in the horizontal plane: surge, sway and yaw. These movements are related to the forces in the fenders and mooring systems that tend to counteract the movements or displacements of the ship from its equilibrium position. It is the virtual mass of

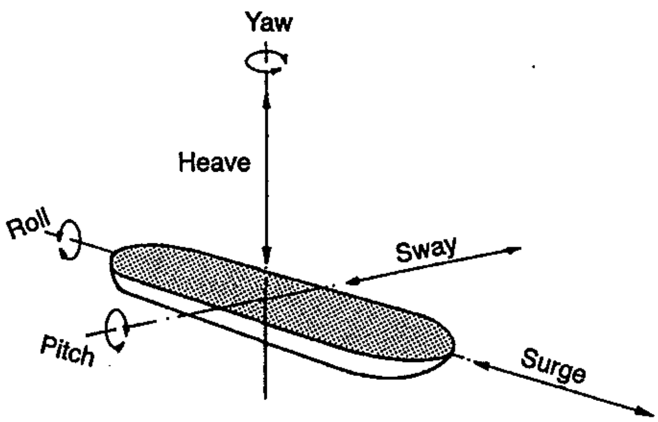


Fig. 4.21. Types of ship's movements

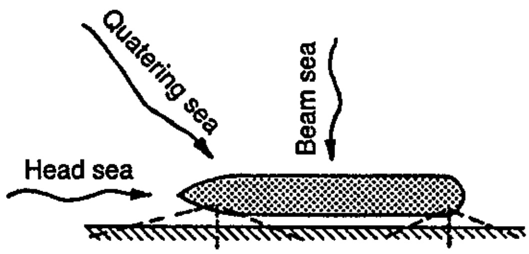


Fig. 4.22. Wave directions

the ship in relation to the fenders and mooring system that governs the natural period of a moored ship.

(b) Movements in the vertical plane: roll, pitch and heave. These are the natural movements of a free-floating ship and are almost unaffected by the fenders and mooring systems.

The ship at berth can be exposed to the following wave directions as shown in Fig. 4.22, and the combination of long and short waves as shown in Figs 4.23 and 4.24. Waves with short periods will more or less only affect small ships while long-period waves will affect more or less all ships.

The wave system is mainly responsible for the unacceptable ship movements and forces in mooring systems. In harbours, for fishing boats or small ships, the shorter periodic waves (less than about 6–8 s) normally determine the berthing and acceptable movement conditions for the ship. For larger ships, it is the longer periodic waves (above 20 s) with a wavelength of about 5000–8000 m and a wave slope of 1 in 2000 to about 1 in 3000, which can cause serious movements and forces in the mooring systems. The reason for this is the risk of the resonance of the long periodic waves having periods of the same magnitude as the natural periods of large moored ships. The ship movement can therefore increase significantly if the periods of the driving forces are in the same range as the natural periods of the moored ship.

The physics of a moored ship is a complex system because the stiffness characteristics are not symmetrical since the fenders are usually

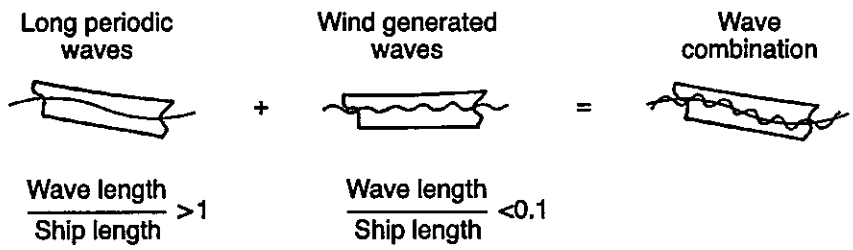


Fig. 4.23. Combination of waves

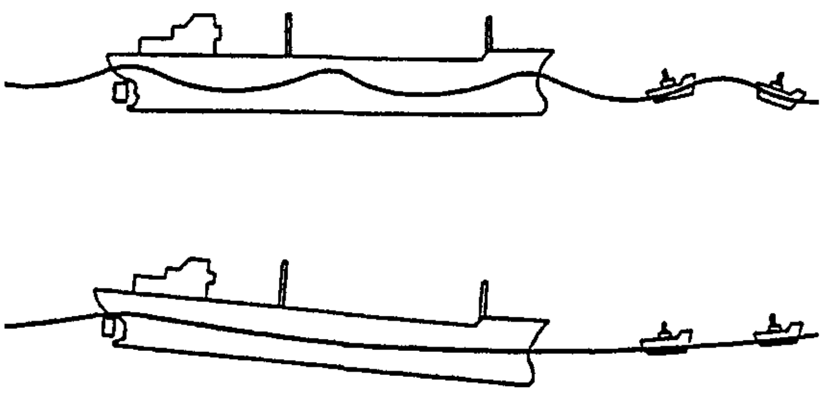


Fig. 4.24. Waves affecting different sizes of ships

stiffer than the moorings, and the deflection characteristics of the fenders and mooring lines are not linear. In addition to the moored ship itself, the wind and wave forces could be time varying and irregular. For a traditional mooring system, a typical natural period for a large ship would be 1 min or more. Therefore, for the purpose of estimation of port operation rate based on acceptable ship movements, one must know the amount of ship movements against the expected wind, wave and current conditions for a ship moored to any kind of mooring system. For the estimation of the ship movements a numerical simulation method or a model test will be the best solution.

Wind forces normal to a moored ship may either press the ship against the fenders or push it away from the fenders. These situations are important to the free movements of a ship at berth, because the ship movements are generally larger when the ship is not in contact with the fender system.

For a ship moored at a berth with a strong current, either parallel or at an angle to the ship, the current may create long periodic ship movements. These periodic movements will depend on the stiffness of the fender and mooring system, the inertia of the ship around the centre of gravity of the virtual mass of the ship, the moment of the respective forces around the centre of gravity and the current velocity. These movements occur when the moored ship is dynamically unstable to lateral displacements and when the current velocity exceeds a velocity of around 1 m/s.

For ships moored at an offshore terminal, such as single-buoy mooring, the movements in the vertical plan, roll, pitch and heave caused by ocean waves will be the critical movements for the terminal's operations.

Table 4.3. Maximum significant wave heights

Ship at berth	H_s at berth
Marinas	0.15
Fishing boats	0.40
General cargo (<30 000 dwt)	0.70
Bulk cargo (<30 000 dwt)	0.80
Bulk cargo (30 000–100 000 dwt)	0.80–1.50
Oil tankers (<30 000 dwt)	1.00
Oil tankers (30 000–150 000 dwt)	1.00–1.70
Passenger ship	0.70

One of the main criteria for the evaluation of a good harbour is whether the ship has sufficiently calm conditions when moored at the berth. It is therefore important to establish realistic criteria for acceptable wave heights and movements at the berth. The acceptable wave heights and movements of a ship at a berth will depend on the elastic properties of the fenders and the mooring system of the ship, the type of ship, the methods used for loading and unloading, the orientation of the berth with respect to the current and wave directions, the wave period and the ship's natural period of oscillation.

The combination of all the physical factors that affect ship movements when at berth are difficult to handle mathematically. Another complicating factor is the human factor, for example the captain's experience, his evaluation of a given critical situation, willingness to take risks, etc. Therefore, the most reliable method of predicting a ship's response to the wave action is to build a physical model of the harbour and the berth. In a physical model the many interactions can automatically be built into the model. For example, current flowing against the waves tends to make the waves steeper and sometimes break, whereas current flowing with the waves tends to reduce the wave height.

As a general guideline the following maximum significant wave heights H_s (in metres) for a head sea, as shown in Table 4.3, have generally been assumed as acceptable for ships at berth.

The table shows that the generally acceptable wave height increases with increasing ship size. The acceptable wave height is highest for a head sea and lowest for a beam sea. The figures can generally be accepted for wave periods up to 10 s. For longer wave periods the figures must be reduced.

In an article in PIANC Bulletin No. 56, Mr H. Velsink, of the Netherlands, published, as shown in Table 4.3, maximum significant

Table 4.4. Maximum significant wave heights for different wave directions

Type of ship	Limiting wave height H_s in metres	
	0° (head on or stern on)	45°–90°
General cargo	1.0	0.8
Container, ro/ro ship	0.5	
Dry bulk 30 000–100 000 dwt loading	1.5	1.0
Dry bulk 30 000–100 000 dwt unloading	1.0	0.8–1.0
Tankers 30 000 dwt	1.5	
Tankers 30 000–200 000 dwt	1.5–2.5	1.0–1.2
Tankers 200 000 dwt	2.5–3.0	1.0–1.5

wave heights H_s for different wave directions before loading and unloading operations have to be stopped. The values refer to the heights of residual deep-water waves with periods in the range of about 7–12 s.

Locally wind generated waves inside a harbour will generally have shorter periods and will therefore have relatively little effect on a moored ship. Waves with very long periods, for example seiches, can have disastrous effects at much lower wave heights than indicated in Table 4.4.

The figures given in Tables 4.3 and 4.4 express the wave criteria in terms of maximum acceptable wave heights at berth. A more realistic criterion would, in most cases, be an expression of the maximum tolerable movement of the ship itself relative to the berth that the mooring system and the cargo handling equipment can tolerate.

The figures for the maximum acceptable significant wave heights must, therefore, always be checked against the ship movement, which can be accepted by the loading and unloading systems. A movement of up to ±2.00 m along the berth is usually acceptable for oil tankers if allowed for in the design of the loading system, but, for the same type of movement along the berth (surge), only ±0.20 m can be accepted for gas tankers. Among the most sensitive berths are the berths for gas tankers, lo/lo container ships and ro/ro ships due to the safety requirements under the loading and unloading operations.

Figure 4.25 shows the relative importance of linear and angular movements of the ship for the safety and efficiency of operations at berth. As shown, the most dangerous movements are the movements in the horizontal plane (surge, sway and yaw), which could break the

	Movement in					
	Horizontal plan			Vertical plan		
	Surge	Sway	Yaw	Roll	Pitch	Heave
VLCC	••	••	••	•	•	•
Cool bulk	•••	••	••	•	•	•
Ore bulk	•••	••	••	•	•	•
Grain bulk	•••	••	••	•	•	•
Supply	•••	••	••	••	••	••
General cargo	•••	••	••	••	••	••
LPG	•••	•••	•••	••	••	••
LNG	•••	•••	•••	••	••	••
Lo/lo	•••	•••	•••	•••	••	••
Ro/ro	•••	•••	•••	•••	•••	••

• = Less important ••• = Most important

Fig. 4.25. The relative importance between the different ship movements

ship loose from the berth. From a safety point of view these movements should therefore be minimized, and the possibility of the build up of a resonant system must be avoided. Some of these movements are more dangerous to safety than others, due to the different types of ships, for example pitch is dangerous to the operation of ro/ro ships, while it will hardly affect the operation of a VLCC ship.

The ship behaviour at berth should always be considered from an operational point of view for this reason. For all ro/ro operations, where trucks are moving over ramps in either the bow or the stern, any movement beyond a couple of centimetres can be of danger to the safety of the operations. It may, therefore, generally be concluded that horizontal movements — surge, sway and yaw — are almost equally dangerous to safety, but that surge is usually the most damaging to operational efficiency.

Therefore, in order to improve the safety and the efficiency of the operations at berth, one must design the mooring and fender system so that the ship can lie as still as possible while at berth to avoid the build-up of inertia in the system. The fender system shall not only be as energy-absorbing and non-recoiling (high hysteresis effect) as possible but, together with the mooring system, provide a damping effect on all movements, and especially by friction between the fender and the ship against surge movements. The ideal system is the one which, by proper tension in pertinent mooring lines, makes the ship rest against the fenders and limits all movements to the smallest as possible. This system requires very strong and safe breast lines. The line material may be steel or synthetic, but if fenders are of a recoiling type, ropes have to be as rigid as possible. That means, combinations of synthetic ropes and non-recoiling fenders are ideal.

The term **'acceptable ship movement'** can be more precisely specified in accordance with the following three main aspects:

(a) Safety limits: If the ship's movement exceeds a certain value this will or can result in damage to the ship, moorings, port installations, etc. This limit is usually specified by an upper limit in the mooring system: mooring lines or mooring winches. The degree of ship movement allowed will be a function of the stiffness in the mooring system due to the fact that a soft system will allow more movement than a stiff system before the mooring loads reach their safety limit.

(b) Operational limits of ship movement: There is a limit to ship movement beyond which the operations of loading and unloading of the cargo can no longer be efficiently or safely performed. The amount of ship movement allowed will depend upon the type of ship, for example an oil tanker, general cargo ship, container ship, etc. Generally, the larger the ship the less it will respond in horizontal movements — surge, sway and yaw — to the primary wave system. The amount of vertical movement limit will for upward movement be a minimum when the ship is in ballast at high tide, while the limit to downward movements will be a minimum when the ship is loaded at low tide.

In the case of container ships, sudden horizontal movements of 1.0 m can significantly lower the rate of loading and unloading. For oil tankers practical limiting movements would be about ±2.0 m for surge and 1.0 m for sway off the fender. For loading and unloading of gas tankers the low temperatures of the gas can cause ice to form by condensation at the joints of the loading arms, and also, because the gas is considered to be more dangerous than oil, the movements considered acceptable are lower than those for oil tankers. In the case of ro/ro ships there are quite stringent limits on the amount of movement allowed.

(c) Different opinions of the harbour authorities and ship operators: These opinions may be expected to vary considerably, but it appears that surge, yaw and roll are considered to be the most dangerous movements in head, quartering and beam seas respectively. This is perhaps due to the fact that hydrodynamic damping is less for these three movements compared with sway, pitch and heave. It is generally felt that for ships above 40 000 ton displacement the surge should not exceed 1.0 m and the yaw and roll movements should not exceed 0.5 m. For smaller ships it appears that slightly larger movements of up to 1.0 m are considered acceptable for surge and yaw, while roll movements of up to 0.7 m are thought tolerable. These figures take no account of the speed of the ship movement, so that an oscillatory movement of 1.0 m completed in 10 s will appear far more alarming than the same movement completed in say 1 min. The acceptable amount of long period surge, sway and yaw movements of large ships may therefore be in excess of the figures given above.

In general the acceptable movements of the ship due to safety-limit criteria will exceed the values due to the limiting degree of ship movement criteria which, according to the very little published information, again will exceed the values given by the opinions of the harbour authorities and the ship operator's criteria.

PIANC Working Group 24 gives, in Table 4.5, the following recommendations, which more or less agree with the research carried out by a joint Nordic group for the different types of ships.

Ir Erik D'Hondt gives very interesting comments to the PIANC report for acceptable movements for container ships during loading and unloading due to the small tolerances in the cell guide location in a container ship. Based on the cell tolerances, he advices that the maximum container ship movements to allow unimpeded cargo handling should be:

(a) Pitching: 0.4° with respect to the horizontal plane.
(b) Rolling: 0.24° with respect to the horizontal plane.
(c) Combined pitching and rolling: 0.45° with respect to the horizontal plane.
(d) Heaving: maximum amplitude 20 cm with respect to point of rest and maximum speed 7.5 cm/s in the most affected cell fore or aft. Most movement calculations relate to the centre of gravity.

Ranges for maximum allowable sudden movements in metres for different types of ships larger than 200 m at berth during loading

Table 4.5. Recommended motion criteria for safe working conditions

Ship type	Cargo-handling equipment	Surge (m)	Sway (m)	Heave (m)	Yaw (°)	Pitch (°)	Roll (°)
Fishing vessels	Elevator crane	0.15	0.15	–	–	–	–
	Lift-on-lift-off	1.0	1.0	0.4	3	3	3
	Suction pump	2.0	1.0	–	–	–	–
Freighters, coasters	Ship's gear	1.0	1.2	0.6	1	1	2
	Quarry cranes	1.0	1.2	0.8	2	1	3
Ferries, ro/ro	Side ramp[2]	0.6	0.6	0.6	1	1	2
	Dew/storm ramp	0.8	0.6	0.8	1	1	4
	Linkspan	0.4	0.6	0.8	3	2	4
	Rail ramp	0.1	0.1	0.4	–	1	1
General cargo	–	2.0	1.5	1.0	3	2	5
Container vessels	100% efficiency	1.0	0.6	0.8	1	1	3
	50% efficiency	2.0	1.2	1.2	1.5	2	6
Bulk carriers	Cranes	2.0	1.0	1.0	2	2	6
	Elevator/bucket-wheel	1.0	0.5	1.0	2	2	2
	Conveyor belt	5.0	2.5	–	3	–	–
Oil tankers	Loading arms	3.0[3]	3.0	–	–	–	–
Gas tankers	Loading arms	2.0	2.0	–	2	2	2

Notes
[1] Motions refer to peak–peak values (except for sway: zero-peak)
[2] Ramps equipped with rollers
[3] For exposed locations 5.0 m (regular loading arms allow large movements)

operations for wave periods between 60 s and 120 s are shown in Table 4.6.

The purpose of Table 4.5 is to quantify and optimize how often different types of ship will be able to use the berth safely. This will be an important input to the evaluation of the operational availability of the berth.

Table 4.6. Ranges for maximum allowable sudden movements

Type of ship	Surge (m)	Sway (m)	Heave (m)	Yaw (°)
Oil tanker	±2.0	+0.5	+0.5	1.0
Ore bulk (crane unloading)	±1.5	+1.0	±0.5	0.5
LNG tanker	±0.2	+0.1	±0.1	0.5
Container	±0.5	+0.3	±0.3	0.5
Ro/ro (side)	±0.3	+0.2	±0.1	0.2
Ro/ro (bow or stern)	±0.1	0.0	±0.1	0.2

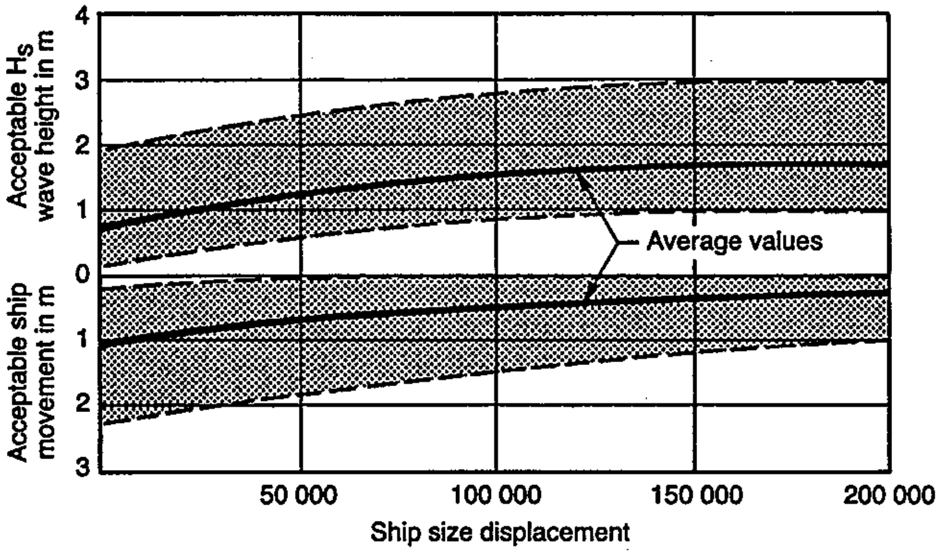


Fig. 4.26. Acceptable wave heights and ship movements at berth

Investigations show that the acceptable significant wave heights increase for increasing ship sizes and that the acceptable ship movement relative to the berth decreases for increasing ship sizes, as shown in Fig. 4.26.

A joint Nordic group involving Denmark, Finland, the Faroe Islands, Iceland, Sweden and Norway published the report *Criteria for Ship Movements in Harbours*. The purpose of this project was to determine criteria for acceptable movements of moored ships in a harbour under working and safe mooring conditions. The project primarily concentrated on assessment of criteria for fishing boats, coasters, container ships and ferries.

For the working conditions during loading and unloading operations, the Nordic group has suggested that the maximum ship movements for working conditions should not be higher than shown in Table 4.7. The figures assume that the occurrence frequency of critical ship movements for fishing boats, coasters and container ships should be less than 1 week/year (2 per cent of the time), and for ferries less than 3 h/year (0.3 per cent of the time).

For the safe mooring conditions only, the Nordic group suggested that the maximum ship movements should not be higher than shown in Table 4.8. The figures assume that the ships are reasonably well moored and the berth structures are well equipped with fenders. For the berth to be acceptable, the frequency of the movements shown below should be less than 3 h/year (0.3 per cent of the time).

The transhipment operations for gas tankers, containers and ro/ro cargo are very sensitive to ship movements and can lead to considerable

Table 4.7. Limiting criteria for ship movements under working conditions

Type of ship	Surge (m)	Sway (m)	Heave (m)	Yaw (°)	Pitch (°)	Roll (°)
Fishing boats						
L_{OA} = 25–60 m						
Lo/lo	±0.75	+1.5	±0.3	3–5	4	3–5
Elevator crane	±0.08	+0.15	–	–	–	1.5
Suction pump	±1.5	–	–	–	–	–
Coasters						
L_{OA} = 60–150 m						
Ship crane	±1.0	+1.5	±0.5	1–3	1–2	2–3
Berth crane	±1.0	+1.5	±0.6	2–4	1–2	3–5
Container ships						
L_{OA} = 100–200 m						
90–100% efficiency	±0.5	+0.8	±0.45	0.5	1.5	3
50% efficiency	±1.0	+2.0	±0.6	±0.6	2.5	6
Ferries						
L_{OA} = 100–150 m	–	+0.8	±0.5	1	1	–

downtime, and therefore the location of the terminal should be chosen with special care.

In the case when the ship movements, in the captain's judgement, exceed the safety limit for the ship to stay at the berth, it is important that the ship has time to leave the berth and the possibility to do so. If the assistance of tugboats is required in the unberthing manoeuvre of a ship, the operational conditions for the tugboats must not prevent them from doing their job satisfactorily. This could be the case when the operational limit for safe operation of the tugboats is

Table 4.8. Limiting criteria for ship movements under safe mooring conditions

Type of ship	Surge (m)	Sway (m)	Heave (m)	Yaw (°)	Pitch (°)	Roll (°)
Fishing boats						
L_{OA} = 25–60 m	±0.75	+2.0	±0.5	6	4	8
Coasters						
L_{OA} = 60–120 m	±1.0	+2.0	±0.75	3–5	2–3	6
Velocity						
Size of ship	m/s		m/s	deg/s		deg/s
About 1000 dwt	0.6	–	0.6	2.0	–	2.0
About 2000 dwt	0.4	–	0.4	1.5	–	1.5
About 5000 dwt	0.3	–	0.3	1.0	–	1.0

reached before the safety limit, or the limiting degree of ship movements, is reached.

4.6 Mooring system

The term 'mooring' refers to the system for safely securing the ship to the berth structure. The mooring of the ship must resist the forces due to the most severe combination of wind, current, waves or swells, seiches, tides and surges from passing vessels.

The berth structure should be provided with mooring facilities such as to permit the largest ship using the berth to remain safely moored alongside the berth structure. In addition the berth should be equipped with sufficient mooring points to provide a satisfactory spread of mooring for the different range of ship sizes which could use the berth.

To assist a port authority to plan the mooring arrangements of a ship arriving at a port, the arriving ship should be requested to send information about its mooring equipment prior to arrival. Information concerning type, number and breaking strength of the mooring lines and winch brake capacity is generally requested. For special berths, like gas and oil berths, the mooring diagrams will normally be made available to the ship's captain by the pilot when boarding.

The mooring forces most difficult to predict are those caused by waves acting along the berth lines. Such forces are probably also the most common reason for broken moorings. The forces on the structure when the ship is moored include the tension in the bollards due to wind and/or current trying to move the ship out from or along the berth structure. Other forces are the horizontal pressure caused by wind and current against the berth structure, and vertical forces caused by the ship chafing on the front (fenders) under vertical movements.

The ship mooring system and configuration must effectively restrain the mooring forces expected to be encountered over the design life for the berth structure while preserving both the operational capabilities and the maximum extreme forces expected on the moored ship and the berth structure.

When wind, waves and current hit a ship between the bow or stern and the beam from any quartering direction, they will exert both a longitudinal and transverse force. The resultant wind or current forces will not have the same angular direction as the wind or current itself. For the evaluation of the mooring layout and arrangement one must add the wind, wave and current forces.

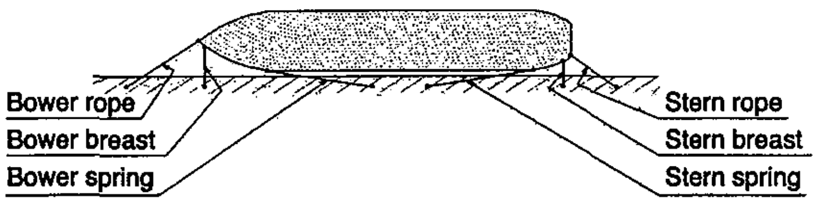


Fig. 4.27. Mooring by hawsers and bollards

A ship coming alongside the berth is usually stopped partly by reversing the engine and partly by retarding using the spring hawser, so that the total design force transmitted to the berth structure through the bollard will at least be equal to the breaking load of the spring hawser. Materials for hawsers are steel wire, manila rope, nylon rope, etc., in other words, different materials implying great variations in the breaking loads and ductility of the various hawsers. Figure 4.27 shows a general mooring arrangement for a ship to a berth by way of bollards.

A fundamental principle is that all forces from the ship normal to the berth front (e.g. wind from land) should be taken by the breast lines, while the forces along the berth front (wind, current, etc.) shall be taken by the spring lines. The forces from wind, waves, and current against a ship at the berth structure will have an influence in the following ways:

(a) **Breast mooring lines**: in the evaluation of the forces, which have to be absorbed by the breasting lines, one must take into account that the topography and that the jetty itself can reduce wind area of the ship. The breast lines are used to reduce the sway and yaw motions, and should be perpendicular to the ship. They should be connected to the bow and stern.
(b) **Spring mooring lines**: in the evaluation of the spring lines' capacities one will normally obtain the full effect from the wind and current along the tanker. The spring lines are used to reduce the surge motion of the ship along the berth front. The spring lines should be as parallel as possible to the berth front. The angle between the berth front and the shipside should be equal to or less than 10°.
(c) **Head and stern lines**: these can be used in addition to the spring and breast lines to reduce the ship's motions.
(d) **Fender system**: in addition to the berthing of the tanker, the fender system must also withstand the forces from wind, waves, etc. against the tanker.

It has been necessary, as has been done in most port engineering standards and recommendations, to specify minimum loadings that

Table 4.9. Bollard load P and approximate spacing

Ships with displacement in tons up to	Bollard load P in kN	Approximate spacing between bollards in metres	Bollard load normal from the berth in kN/m berth	Bollard load along the berth in kN/m berth
2000	100	5–10	15	10
5000	200	10–15	15	10
10 000	300	15	20	15
20 000	500	20	25	20
30 000	600	20	30	20
50 000	800	20–25	35	20
100 000	1000	25	40	25
200 000	1500	30	50	30

the bollards shall be able to resist for ships of various tonnages. Thus the bollards, their dimensions and anchoring, and the berth structure itself shall be designed for a certain minimum loading. The idea is that if a ship has a too strong hawser compared to the design load of the bollard only the latter will break at its footing without the berth structure itself being much affected.

Bollards should be provided at intervals of approximately 5–30 m depending on the size of the ship along the berthing face. The bollard load P and the approximate spacing between bollards should be as shown in Table 4.9.

For larger ships, specific calculation must be carried out to determine the maximum bollard load, taking into account the type of ship and the environmental loading.

Bollard loads are assumed to act in any direction within 180° around the bollard at the seaside, and from horizontally to 60° upwards, as shown in Fig. 4.28.

If the berth structure is exposed a lot to wind and current, the above bollard loads should be increased by 25 per cent. When the ship is moored, the berth structure should be designed for a minimum vertical force of 0.87 times P. The bollard foundation itself should be designed for a force 20 per cent greater than the capacity of the bollard.

Mooring dolphins should be designed for the same loads as the bollards. In addition to the usual berth bollards, storm bollards are often installed behind the apron, designed for twice the above bollard loadings.

If the same bollard accommodates more than one hawser, some standards recommend that the bollard should still be designed for the

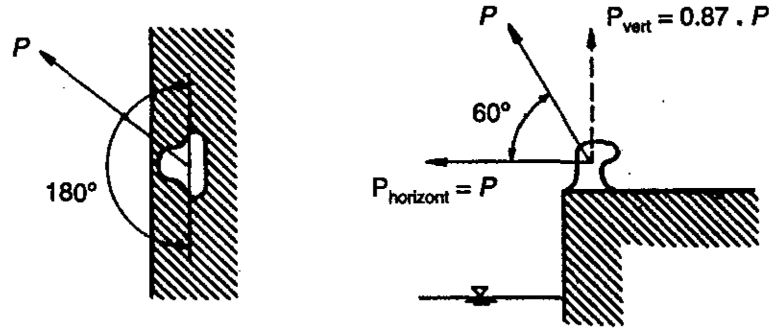


Fig. 4.28. Bollard load directions

tabulated load only. This is because it is most unlikely that all the hawsers are fully loaded and pulling in the same direction at the same time. But it is recommended that if it is possible for two ships to use the same bollard point, one should install either two bollards or one double bollard. Therefore, if two ships use the same single bollard and the first ship leaves, the second ship may be reluctant, in windy conditions, to temporarily slacken its mooring rope to enable the first ship to unberth.

Generally the mooring lines should be symmetrical about the centreline of the ship to obtain an equal load distribution over them. It is important that the normal or transverse forces from the stern and aft mooring lines are symmetrical around the centreline of the ship if it is exposed to ship motion. All the mooring lines and fenders should ideally have the same stiffness. To prevent impulse shock on the mooring lines and to create sufficient friction between the ship and the fenders, the mooring lines should always be kept taut.

To all **mooring points**, mooring lines of the same size and materials should be used. For an oil or gas tanker the mooring lines should be arranged as symmetrically as possible about the centreline of the piping manifold or transverse centreline of the ship. The spring lines should be oriented as parallel as possible to the longitudinal centreline of the ship. The breast lines should be oriented perpendicular to the longitudinal centreline of the ship and as far aft and forward as possible, as shown in Fig. 4.29. All the mooring lines for large tankers should be between 35–50 m.

The breast line horizontal angles should, if possible, be less than 15° between the ship and the shore mooring point. The head and stern line should be about 15°. The spring horizontal angle should be less than 10°.

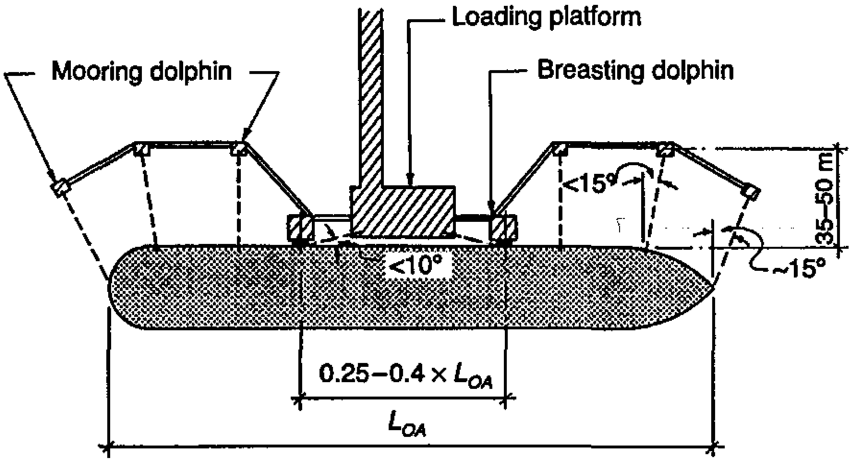


Fig. 4.29. Layout of dolphins for berthing of tankers

The maximum vertical angle of the spring and breast mooring lines should be as small as practical and preferably not exceed 25° from the horizontal throughout the entire range of the ship loading or unloading conditions. These criteria will therefore determine the position of the mooring points. This means that, for example, the breast mooring structures will be located approximately 35–50 m behind the berthing face as shown in Fig. 4.30.

The distance between the breasting dolphins should be approximately 0.3 of the overall length L_{OA} of the ship. If the breasting

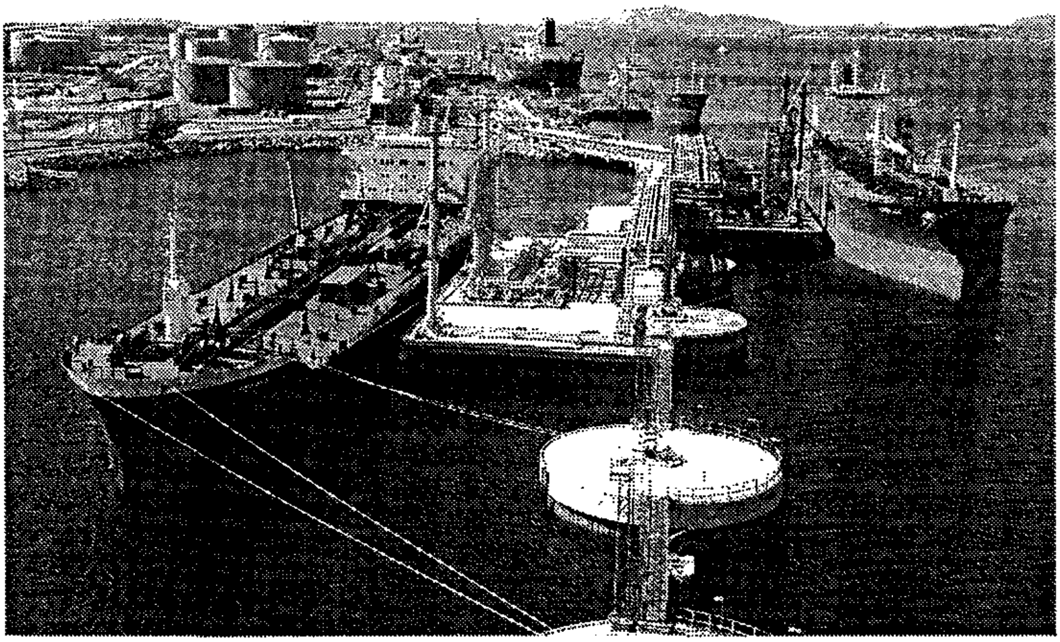


Fig. 4.30. Two tankers moored at the same jetty. Photo courtesy of Øyvind Hagen, Statoil Mongstad, Norway

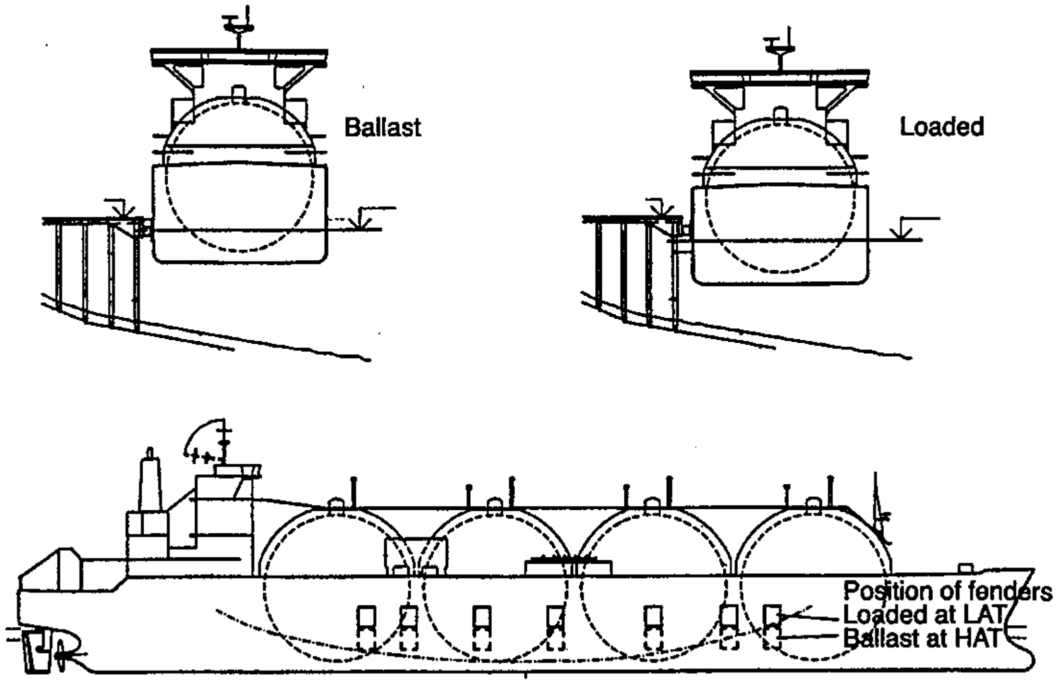


Fig. 4.31. The distance between the breasting dolphins is within the flat bodyline of the ship's side

dolphins should accommodate a range of different ship sizes, the distance should be within 0.25–0.4 of the ship's overall length. It is very important that the distance between the breasting dolphins is within the flat bodyline of the ship's sides, as shown in Fig. 4.31.

The mooring arrangement described above will be suitable for oil tankers and for LNG and LPG tankers with flush deck structures (membrane type, etc.). For cylindrical tank LPG carriers and spherical tank LNG carriers it is usually not practicable to have the main deck winches for the spring lines on the main deck. The spring lines must, therefore, be accommodated from the forward main deck and from the aft as suggested by the Oil Companies International Marine Forum in Fig. 4.32. For more details see the Oil Companies International Marine Forum recommendations.

At a berth structure for oil and gas tankers the mooring line should be fixed to **mooring hooks**, instead of a bollard, with capacities up to 3000 kN, manual or oil hydraulic release devices and remote control systems for quick release of the mooring lines if a ship leaves in an emergency. To simplify the running of the mooring lines from the tankers to the mooring hooks and pulleys, the hooks and pulleys should have motor-driven **capstans** behind the mooring gear to haul in the heavy mooring lines for a double **quick-release hook** (QRH) from Mampaey or equivalent as shown in Fig. 4.33.

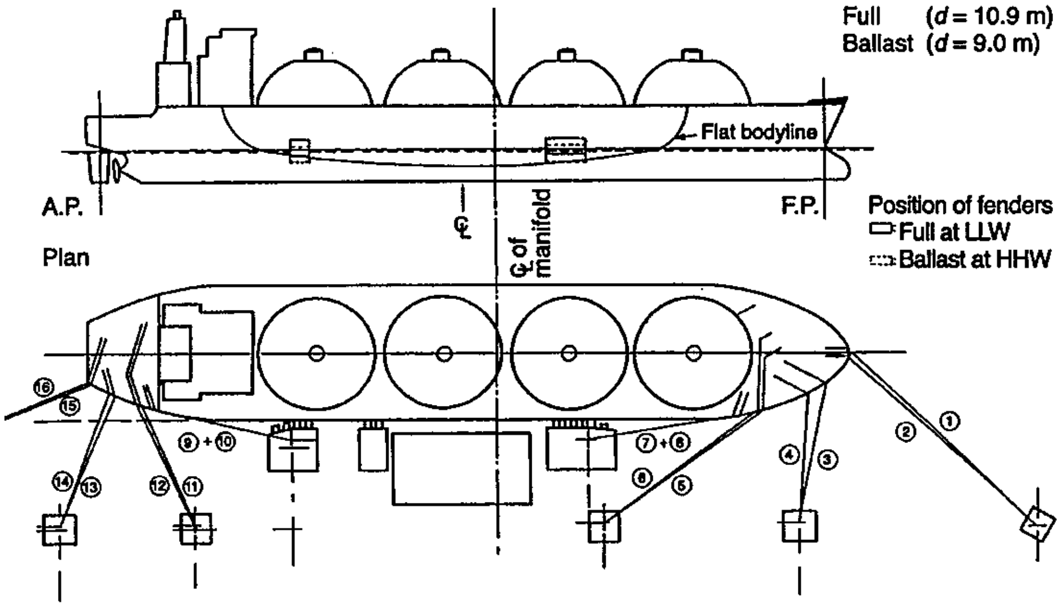


Fig. 4.32. Mooring arrangement for cylindrical and spherical tankers (Note: Circled figures represent mooring lines)

The capstans should have about a 30–40 cm diameter barrel and have a standard line-pull capacity range of 1–3 tonnes, motor geared to give a pull rate of about 25–30 m/min.

For gas or oil berths, each of the mooring points should be equipped with QRHs with capstans. The QRHs would usually have a safe working load (SWL) of 600–2000 kN.

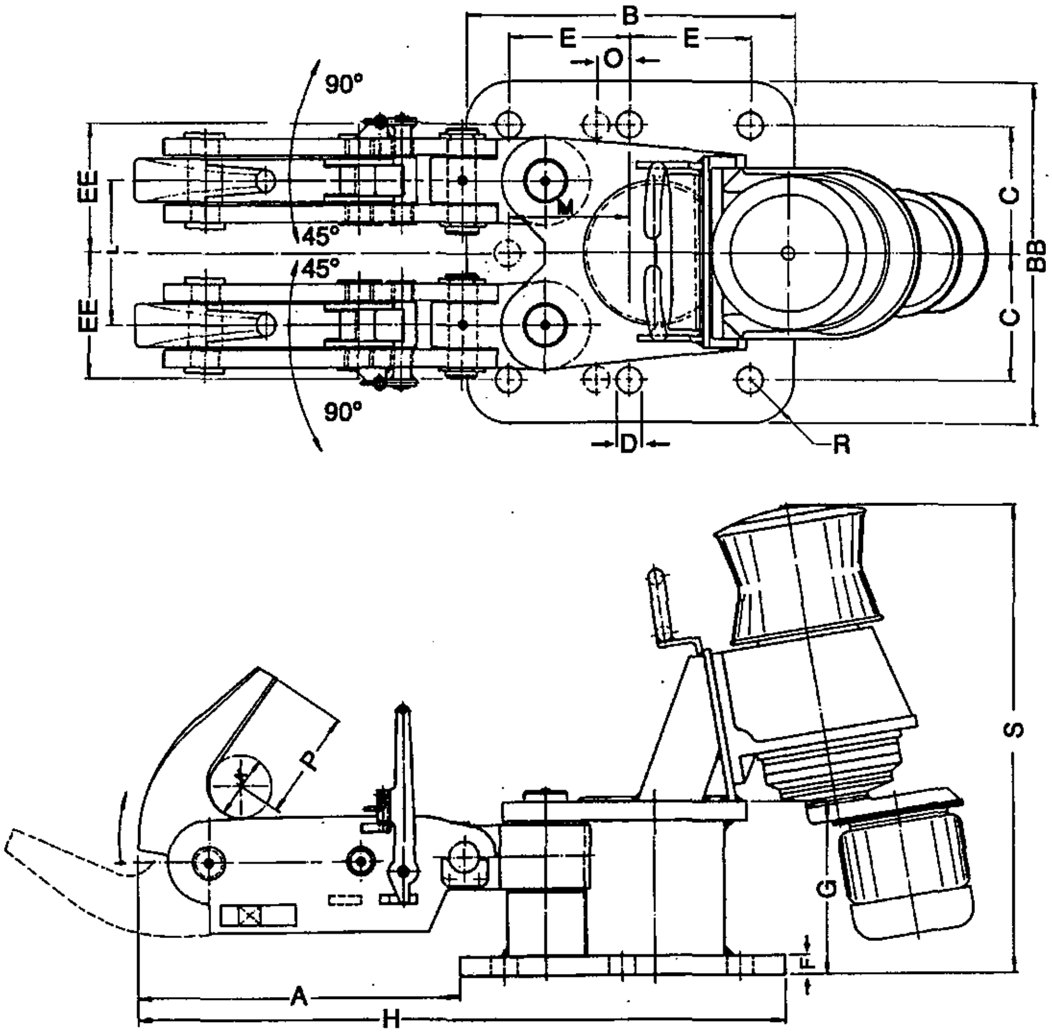
The design load for each hook support structure should be defined as the total number of hooks times the safe working load per hook.

The proof load (PL) is 1.5 × SWL. In addition, a material factor of 1.3 should be used. The QRHs should be provided with a SWL of not less than the minimum breaking load (MBL) of the largest rope anticipated to enable the handling of the mooring of the largest ship.

If required, the QRH locking mechanism can be designed to fail and/or release the mooring line at any predetermined line load. The QRHs can also be equipped with a radio-controlled remote system. The hook should have a built-in safety-locking device to prevent accidental or unauthorized release.

All mooring dolphins or mooring structures should be designed on the principle that the breaking limit of each mooring point or dolphin must be at least 20 per cent greater than the total breaking limit of all **mooring lines** which can be put out to the mooring point. In other words the design load should be 1.2 times the sum of all mooring lines' breaking loads.

Each mooring hook member is usually capable of taking up to three separate mooring lines of approximately 50 mm diameter each. For gas



Explanation: Cap = Capacity mounting base in kN
Wt = Weight in kg; excluding capstan
X = Number and size of HD bolts

Weight capstan ± 450 kg
Dimensions in mm

Size	Cap.	Wt.	A	B	BB	C	CC	D	E	EE	F	G	H	K	L	M	N	O	P	R	S	T	V	W	Y	Z	X
60	1200	850	850	750	780	290		62	275	290	45	402	1600	150	330				250	100	1250						6 × M 56
75	1500	1040	900	750	850	325		70	275	325	45	417	1650	150	350			55	250	100	1250						6 × M 64
100	2000	1450	980	750	925	363		78	275	363	50	437	1730	150	375			75	275	100	1250						6 × M 72
125	2500	1500	980	750	925	363		86	275	363	55	447	1730	150	375			75	275	100	1250						6 × M 80
150	3000	2360	1220	750	1000	400		78	275	400	55	490	1970	150	425	275		50	300	100	1250						7 × M 72
200	4000	3000	1200	850	1000	380		96	305	380	65	562	2050	150	425	250		30	300	120	1250						7 × M 90

Fig. 4.33. A principle layout of a double QRH with capstan

tankers, the Society of International Gas Tanker and Terminal Operators Ltd recommend that multiple hook assemblies should be provided at those mooring pointes where multiple mooring lines are deployed so that not more than one mooring line is attached to each single hook.

The following are types of mooring lines or hawsers that are generally used for the various types of ships:

(a) Freighters and coasters, which are sailing on domestic and or short sea routes, are ships generally less than 10 000 dwt. These ships are generally moored with polypropylene lines.
(b) General cargo ships, ranging from about 5000–10 000 dwt, are generally moored with polypropylene lines. Larger ships are generally equipped with nylon lines and/or steel wires.
(c) Large tankers are generally moored with steel wires and steel wires with nylon tails.
(d) Bulk carriers are mainly moored with synthetic lines and have steel spring lines. All mooring lines are attached to bollards along the berth front rails for loading and unloading of the ship.
(e) Container ships are generally moored with steel spring lines to reduce the surge motions and with polypropylene mooring lines.

The OCIMF recommend that if there is no knowledge of a specific berth geometry, the ship's general mooring line requirement should be based on the maximum components of the environmental forces and assuming an efficiency of 90 per cent of the spring lines and 70 per cent of the breast lines. The necessary number of mooring points should be designed on the basis that the maximum allowable loads in any one of the mooring lines should not exceed 55 per cent of its MBL or 100 per cent of the ship's rated winch-brake capacity.

The wind and current forces on a tanker should at least be designed after the OCIMF recommendation, but one should be aware that other different standards and recommendations, as indicated in Chapter 2, can give other design forces.

The elasticity of the mooring lines will depend on the following:

(a) diameter of the mooring line
(b) length of the line from the ship to the mooring point
(c) material and construction of the line.

The following materials are used in the mooring lines:

(a) Natural-fibre lines are manufactured from manila, sisal, etc. and are the traditional mooring lines. These lines have low load/diameter factors and cannot easily absorb peak loads. Their lifetime is relatively short.
(b) Synthetic fibre lines are manufactured from polypropylene, nylon, etc. Compared to natural fibre they have high load/diameter ratio, are relatively light and are easier to maintain. Their lifetime is relatively long and they are relatively cheap to buy.

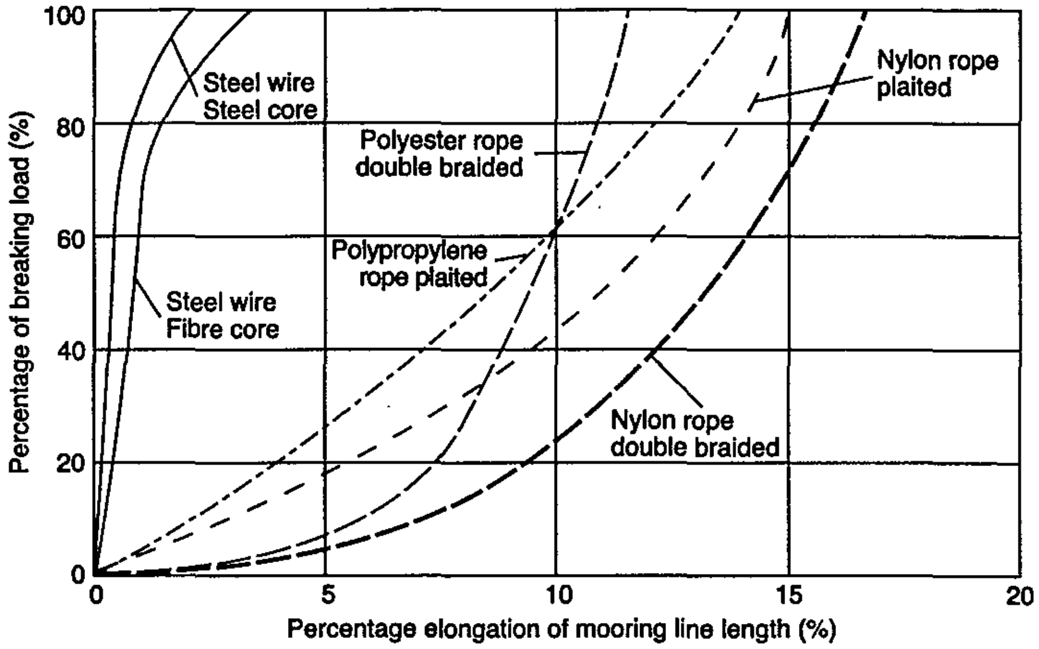


Fig. 4.34. Typical mooring-line force characteristics

(c) Steel wires are stiff, have high load/diameter ratios and low elongation. Their lifetime is long and they are relatively cheap to buy.

(d) Combi-lines are a combination of steel wires with synthetic tails. The tails should not be longer than 10 m. Because of the synthetic tails these lines have excellent shock absorption characteristics. They are particularly used for the mooring of large tankers.

From the mooring point of view it is important that all breast mooring lines and spring lines have the same length and are of the same materials. Mooring line with high elasticity is desirable for ship-to-ship mooring and on mooring for berth subjected to swell but, on the other hand, can cause problems with gantry cranes or loading arms. OCIMF recommends that the safety factor for steel lines should be 1.82 and for nylon tails 2.5. Figure 4.34 shows typical mooring-line force characteristics.

The release of the mooring lines can be done locally at the hook or pulley, or by remote control. A continuous control of the tension of the mooring lines should be kept by remote-reading tension meters. The mooring lines should be adjusted by picking up the slack and re-adjusted by the mooring winches during loading or unloading of the ship. The mooring-line tension must be continuously adjusted, but automatic tension must not be allowed.

For safety reasons, the terminal mooring supervisor should oversee the mooring from the terminal control centre and by regular inspections at the berth, as long as the tanker is alongside. The tanker should be notified if the moorings are not properly maintained and tightened. The mooring lines will require to be tightened due to changes of tide, freeboard or weather, to prevent them from being overloaded or going slack. All moorings on self-tension winches should be secured with winch breaks in the locked position.

The movement of moored ships that have synthetic mooring lines should not exceed the design envelope of the loading arms, hose or gangway structure. Where synthetic tails are used on the end of the wire-mooring lines to reduce dynamic peak loading, these should be examined to ensure that the design envelope is not exceeded.

In deteriorating weather conditions, the ship's captain may have to decide to use additional mooring lines, request stand-by tugboats to hold the ship alongside the berth, or leave the berth for open sea.

After loading or unloading operations are completed, operators should check the berth area for any local restrictions or hazards. The unberthing is monitored by the terminal mooring supervisor from the terminal control centre, but is directed by the ship's captain assisted by the pilot and tugs. It is always the ship's captain who decides when, and in which order, mooring lines will be released.

For design safety, weather conditions and commercial criteria, the operation of oil and gas tankers will generally require that a tanker should be loaded or unloaded in approximately 12 h, so that the tanker can turn around in 24 h.

If a fire occurs, either ashore, at the berth or on the ship, that cannot be extinguished with the fire fighting facilities immediately available, a decision may be taken as to whether the ship should remain at berth or should be removed by tugboats to a safe distance away from the berth.

The **mooring forces** against the tanker from wind, waves and currents are difficult to estimate accurately. In the evaluation of the forces against the berth structure and the forces to the mooring system, it is recommended by OCIMF that the design wind velocity shall be 30 m/s against the tanker, and the current velocity should be between 1.0–1.5 m/s parallel to the tanker. The reason for this is that if the tanker cannot unberth before a storm, the berth structure and the mooring system must be able to absorb all the forces from the storm. In countries with very rough and exposed coastlines, it could be justified, for environmental reasons, that a designed wind velocity of 40 m/s without gust factor be used.

Table 4.10. Forces in kN against tanker in loaded condition

Size of tanker in dwt	Wind 40 m/s against tanker				Current 1.0 m/s along tanker	Wind and current parallel to tanker
	Normal to tanker	Wind 45° to tanker		Parallel to tanker		
		Normal to tanker	Parallel to tanker			
200 000	6403	4246	707	1481	50	1531

As an example the forces during a full storm of 40 m/s without any gust factor acting against a ballasted 200 000 dwt oil tanker, will after the OCIMF recommendations with ship wind area for a 95 per cent confidence limit be as indicated in Table 4.10.

This clearly shows that, with a full storm blowing normal to the tanker at the berth, it will be nearly impossible for the tanker to leave the berth safely even with extended tugboat assistance.

4.7 Visibility

The types of weather condition, that can cause bad visibility are fog, heavy rain and snow. Fog is defined by some standards as a weather condition in which the visibility is less than 1000 m. The combination of heavy snow or rain together with strong wind is considered more difficult for berthing operations than fog, which usually appears in calm weather when the ship is easier to handle.

In general, visibility between 500–1000 m can be acceptable for the manoeuvring and berthing process inside a harbour. If the visibility is less than 2000 m the ship's velocity should be reduced at least to 6 knots for ship sizes above 10 000 dwt. For visibility less than 1000 m it is advisable for safety reasons that all larger ships and oil and gas tankers should have tugboat assistance in restricted areas like main traffic channels, inner harbours, oil terminals, etc.

Regulations in some of the larger ports say that no ship should start manoeuvring within the port area if the visibility is less than three times the ship's overall length. For tugboats with a tow, the ship's length includes the length of the entire tow.

As a general rule, most oil and gas terminals will close for arrival and berthing or unberthing and departure of tankers if the visibility is less than between 1000–2000 m.

4.8 Port regulations

Generally it is the ship's captain who is responsible for the manoeuvring of the ship to and from the berth and the mooring of the ship at berth. The port captain and his staff are responsible for the berth structure itself, the berth and mooring equipment, and that the ship's captain follows all the port regulations. If the ship's captain does not comply with the port safety requirements or the port regulations, the port captain has the right to stop all operations and order the ship off the berth for appropriate actions to be taken by the ship's owners and charterer concerned.

4.9 Availability of berth

Nearly all the items discussed in this chapter will together lead to the total availability or, the opposite, the downtime of the berth, which again can be subdivided into the following two cases:

(a) **Navigational availability**: describes the percentage of time the ship is able to call at the harbour or berth safely from the open sea or ocean.
(b) **Operational availability**: describes the percentage of operational time during which the ship can operate by loading and unloading at the berth.

The availability should not only give the overall availability of the berth per year, but also the availability of the berth for each month. Dependent on the type of berth, the total yearly navigational and operational average availability should not be less than about 90–95 per cent due to the extra cost of waiting time for the ships to call at the port, for example, or the **downtime** should not be more than about 5–10 per cent.

The designer should always try to evaluate the downtime of a berth due to the navigational and operational availability. The acceptable figures for the downtime for a berth would vary according to the type of cargo handled. For an oil and gas berth an approximate limit of 10 per cent downtime on an annual basis is the norm.

The yearly preliminary estimated availability could be calculated for a possible harbour location for oil tankers of between 150 000–300 000 dwt, as illustrated in Table 4.11.

In general, the downtime calculation of a berth should involve the determination of critical wind, wave and current conditions that could cause unacceptable ship motions or mooring line loads, and/or an inability to operate (e.g. because of the berth crane equipment).

Table 4.11. Yearly estimated availability

Non-availability due to	Frequency of estimated percentage downtime
Navigation	
Ice problems	–
Excessive current	–
Wind above 10 m/s, which will stop the manoeuvring of the tanker	4.5
Waves above 1.5 m, which will stop tugboat assistance	0.2
Swell and long period waves	–
Visibility less than 1000 m	0.2
Tugboat non-availability	0.05
Operation	
Stop in loading operation due to wind above 20 m/s	0.2
Excessive ship movements at berth	0.1
Maintenance on berth structure	0.5
Average estimated percentage downtime	5.75

These critical conditions would define the boundary between the acceptable operations and the availability or downtime of a berth.

The estimated accuracy for this type of availability or downtime evaluation is about ±1–3 per cent. In this example it has not been taken into consideration that some of the non-availability factors can act together and therefore reduce the sum of the yearly estimated percentage of downtime obstacles.

The yearly berth availability should generally be approximately 95 per cent, and not less than approximately 85 per cent at any particular month of the year. But this will depend on the type of traffic and the importance of the cargo traffic.

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5

Impact from ships

5.1 General

In this chapter the berthing forces that can arise between a berth structure and berthing ship will be discussed. The berthing forces transmitted to the structure will consist of impact loads normal to, and frictional loads parallel to, the berthing face.

While the vertical loads on a berth from dead weight, live load, crane loads, etc. can be determined very accurately, it could be very difficult to evaluate the horizontal loads caused by ships' impacts. The size and velocity of ships when berthing, the manoeuvring, direction and strength of current, wind and waves at the berth are factors that often escape an exact quantification and therefore tend to complicate the correct calculation of ships' impact forces.

The following design criteria should be used in the calculation of the berthing and mooring energies and in the selection of the fender system to be used:

(a) the design codes and regulations
(b) the desired design life of the berth structure and the safety factors to be used
(c) the design berthing ship and the ship's allowable hull pressure
(d) the design berthing velocity both under normal and abnormal conditions.

Based on normal berthing procedures, the berthing energy and the impact forces from the berthing ship against the berthing structure can be estimated from one of the following:

(a) the theoretical method
(b) the empirical method
(c) the statistical method.

The ship's berthing energy is proportional to the virtual mass of the ship and to the square of the approach velocity, and is reduced according to the rotation by the eccentric berthing when the ship hits the berth structure at a distance from its centre of gravity.

5.2 The theoretical or kinetic method

The theoretical method is based on the general basic kinetic energy equation due to the impact of a ship on a berthing structure:

$$E = 0.5 \times M_v \times V^2 = 0.5 \times (M_d + M_h) \times V^2$$

where

E = the kenetic energy in kN m
M_v = the virtual mass in ton, which equals the displacement of the ship M_d plus the hydrodynamic or added mass moving with the ship M_h
M_d = the mass of the design ship (the displacement in tonnes) should, after the PIANC Fender 2002, be based on the 95 per cent confidence level. For the guidance on the displacement for the various ships see Chapter 20 about 'ship dimensions'. In most cases the ship fully loaded displacement should be used for the fender design
M_h = when the ship is moving through water, there is also the movement of a volume of water around the ship, which is entrained with it, to be considered. When the ship comes to a stop this additional mass of water will continue to move and press the ship against the berth. This additional mass of water is also known as the added mass or as the hydrodynamic mass
V = the velocity in m/s of the ship's normal to the berth line. After the PIANC Fender 2002, the 50 per cent confidence level should be used.

For all berth structure design, the displacement for a fully loaded ship should be used, except where the berth will be used exclusively for the export of cargo, then the displacement of the ship and the draft may be reduced to the actual value for the ship when berthing, but not less than the ballast displacement.

Out of the total kinetic energy of the ship, the fender system must absorb:

$$E_f = C \times (0.5 \times M_d \times V^2)$$

where the adjusting factor or berthing coefficient is $C = C_H \times C_E \times C_C \times C_S$.

Hydrodynamic mass factor C_H

The hydrodynamic or added mass factor C_H allows for the movement of the water around the ship to be taken into account in the calculation of the total berthing energy of the ship by increasing the mass of the system.

$$C_H = \frac{M_d + M_h \times C_{HR}}{M_d} = 1 + \frac{M_h \times C_{HR}}{M_d}$$

where

- C_H = the hydrodynamic or added mass factor
- M_d = the displacement of the ship
- M_h = the hydrodynamic or added mass
- C_{HR} = the reduction factor due to the ship moving at an angle to its longitudinal axis. In principle, the reduction factor C_{HR} for the hydrodynamic mass of a ship moving normal to the berth line in open water will be 1.0, but for a ship moving along its longitudinal axis in open water it can be assumed to be about 0.1.

Over the years different formulas for the hydrodynamic or added mass factor have been suggested as shown in Table 5.1.

where

- ρ = the specific gravity of sea water (10.3 kN/m^3)
- D = the draft of the ship
- B = the width of the ship
- L = the length of the ship
- H = the water depth
- M_d = the displacement of the ship
- C_B = the block coefficient $= \dfrac{M_d}{\rho \times L \times B \times D}$

The displacement is the product of the length between perpendicular L_{BP} times the draft D times the width B times the block coefficient C_B. The PIANC Fender 2002 recommends the following block coefficient

Table 5.1. Formulas for hydrodynamic or added mass factor

Author	Year	Type of test and comments	Formula for C_H
Stelson (PIANC, 2002)	1955	Model test	$1+\dfrac{1/4\times\pi\times\rho\times D^2\times L}{M_d}=1+\dfrac{\pi\times D}{4\times C_b\times B}$
Grim (PIANC, 2002)	1955	Model test	$1.3+1.8\times\dfrac{D}{B}$
Saurin (PIANC, 2002)	1963	Full-scale observation and model test	1.3 (Mean value) 1.8 (Safe value)
Vasco Costa (PIANC, 2002)	1964	Model test	$1.0+2.0\times\dfrac{D}{B}$
Giraudet (PIANC, 2002)	1966	Model test	$1.2+0.12\times\dfrac{D}{H-D}$
Rupert (PIANC, 2002)	1976	Full-scale observations	$0.9+1.5\times\dfrac{D}{B}$
Ueda (PIANC, 2002)	1981	Full-scale observations	$1+\dfrac{1/2\times\pi\times\rho\times D^2\times L}{M_d}=1+\dfrac{\pi\times D}{2\times C_b\times B}$

C_B to be adopted if lacking other data:

Container ship	0.6–0.8
General cargo ship and bulk carriers	0.72–0.85
Tankers	0.85
Ferries	0.55–0.65
Ro/ro ship	0.7–0.8

Professor F. Vasco Costa of Portugal assumes that the ship moves sideways to, for example, a berth or rotates about its centre of gravity. The Professor F. Vasco Costa formula is valid if the keel clearance is more than $0.1 \times D$ and the ship velocity is more than 0.08 m/s. If the ship moves along its longitudinal axis, Professor F. Vasco Costa assumes that $C_H = 1$.

The formulas of professor F. Vasco Costa and Professor Shigeru Ueda of Japan are nowadays presumably the most used formulas for calculation of the hydrodynamic mass factor.

The exact value of the hydrodynamic mass is very difficult to determine. Investigations and researches have shown that the hydrodynamic mass will vary with the shape of the ship, the under keel clearance, the ship velocity and the water depth. The hydrodynamic mass usually varies between about 25 and 100 per cent of the displacement of the ship. Generally it is recommended that for a water depth of 1.5 times the draft of a ship or more, C_H be taken to be 1.5. When the water depth is only 1.1 times the draft of the ship, C_H is taken to be 1.8.

Eccentricity effect C_E

The eccentricity factor C_E is due to the consideration of the energy dissipation which arises from the rotational motion after berthing around the contact point either at the bow or at the stern.

$$C_E = \frac{i^2 + r^2 \times \cos^2 \phi}{i^2 + r^2}$$

where i is the ship's radius of inertia, generally between 0.2 and 0.25L, r is the distance of point of contact from the centre of mass, ϕ is the angle as shown in Fig. 5.1.

Figure 5.1 shows C_E as a function of the angle ϕ and the ratio r/L when $i = 0.2L$.

If ϕ is 90° the equation will be

$$\frac{1}{1 + \dfrac{r^2}{i^2}}$$

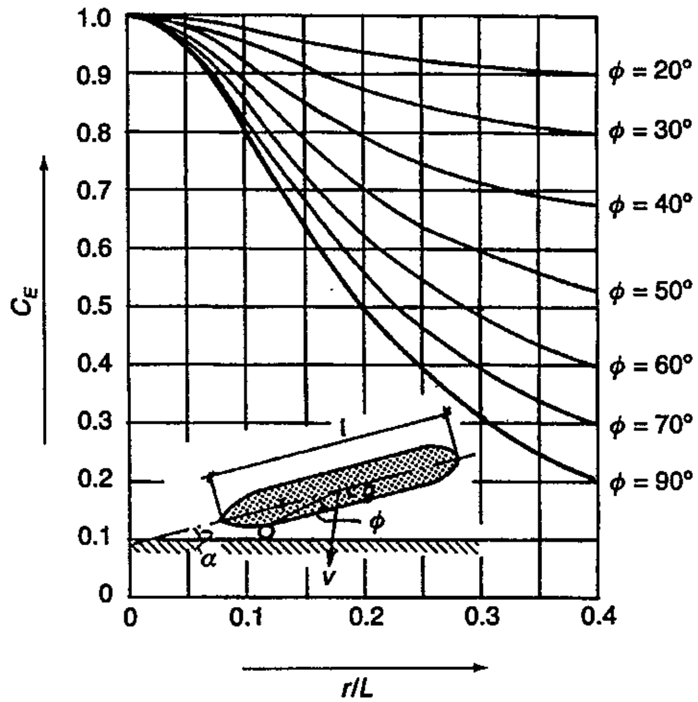

Fig. 5.1. Eccentricity effect C_E as function of ϕ and r/L

Figure 5.1 indicates that the way ships come alongside is a very important matter. This should therefore be studied as closely as possible. The value of $\phi = 90°$ may give too favourable values.

Normally either the ship will come alongside under its own power and with an angle to the berth line, or the ship will have nearly stopped outside the berth structure and be carefully manoeuvred towards the structure with the help of current, wind and/or waves. When the ship is berthing at an angle, it will usually make contact only with one fender.

Figure 5.1 further indicates that the value of C_E also depends on which part of the ship the impact comes. Usually the berthing or angle of approach, which is the angle between the berth line and ship, will be about 1–5° if the ship is berthing with tugboat assistance. If the berthing manoeuvring is done without tugboat assistance the berthing angle will usually be less than about 10–15°. Then the distance between the gravity centre of the ship and the point of the impact, r, is about $0.25\text{–}0.35 \times L$. If now the angle ϕ is approaching 90°, there will be a minimum amount of impact energy hitting the berth structure. For a continuous fender system, C_E is generally taken between 0.5 and 0.6 and for berth structures with, for example, individual breasting dolphins, C_E is taken to be between 0.7 and 0.8.

If the ship comes alongside parallel to the berth front, i.e. $\alpha = 0°$, the ratio r/L also approaches 0, and one will get the maximum amount of impact energy. On the other hand, the part or length of the ship hitting the structure is now far greater, implying that the energy to be absorbed per lin m of berth structure will be less than in the above case.

Therefore, if one assumes the most favourable values of, respectively, α, ϕ and r/L, one would be able to theoretically find a very moderate impact energy. But in practice, however, manoeuvring will deviate from the ideal assumptions, and it is advisable to choose realistic values.

Water cushion effect C_C

The water cushion effect, C_C, is 0.8–1.0 at solid and open berths respectively. If the bottom is sloping steeply under the berth, the resistance from the water will increase when the ship comes near the berth front. This is particularly true at solid berths (e.g. steel sheet-pile structures where the water between ship and berth has to be squeezed aside before the ship can touch the berth structure). For open berth structures, where there is usually an easy way out for the water between the berth and the ship, the water cushion effect will hardly occur.

Softening effect C_S

This factor is determined by the ratio between the elasticity and/or the flexibility of the ship's hull and that of the fender system or berth structure. Therefore, part of the berthing kinetic energy will be absorbed by elastic deformation of the ship's hull and or flexibility of the berth structure. For a small ship the C_S is generally taken to be 1.0. For hard fenders and larger ships, e.g. large tankers or flexible wood piers, the C_S softening effect is 0.9–1.0.

British Standard BS 6349, Part 4 says that a hard fendering system can be considered as one where the deflection of the fenders under impact from ships for which the fenders are designed, is less than 0.15 m.

Approach velocity v

It appears from the above that sophisticated calculations to establish the magnitude of the adjusting factor or the berthing coefficient C are not justified if only approximate values of the velocity v can be found. The approach berthing velocity v is the most influential variable in the calculation of the berthing energy. The approach velocity is defined as the ship' speed at the initial berthing contact, measured perpendicular to the berth line. After the PIANC Fender 2002 the mean velocity value v should be taken to be equivalent to the 50 per cent confidence level.

Determining the correct value of the approach velocity, which is the most significant parameter in the energy equation, is very difficult, but since it appears in the energy formula in the second power one must try to find as accurate a value as possible. This is illustrated with a berthing coefficient $C = 1.0$ in Fig. 5.2.

The actual berthing approach velocity will be influenced by a large number of factors such as the following:

(a) The experience of the crew during the berthing operation.
(b) The influence of the wind, waves and current around the berth structure.
(c) Is the navigation approach to the berth easy or difficult, and is the approach channel equipped with navigational aid systems?
(d) Is the ship equipped with bow thrusters and does it have good manoeuvrability, or is it necessary to use tugboat assistance?
(e) Type of ship, e.g. container ship, tanker, general cargo ship, and the windage area of the ship, etc.

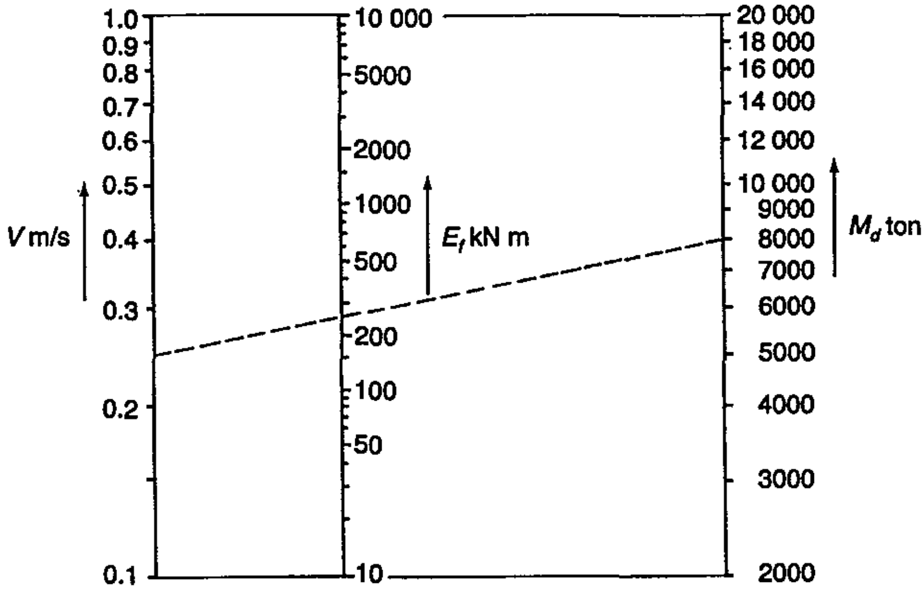

Fig. 5.2. The fender energy E_f with berthing coefficient C = 1.0

Normally, smaller ships have greater velocities when hitting the berth structure than larger ships. Figure 5.3 suggests velocities for different sizes of small and medium ships, berthing **without tugboat** assistance and related to the various weather and manoeuvring conditions.

The ship sizes shown in Fig. 5.3 and Table 5.2 are taken to the 50 per cent confidence level. In case the berthing manoeuvre takes place without tugboat assistance, the figure below should be increased considerably.

Due to safety reasons and to reduce the probability of damage to the fender systems, PIANC recommend that for the design of fender

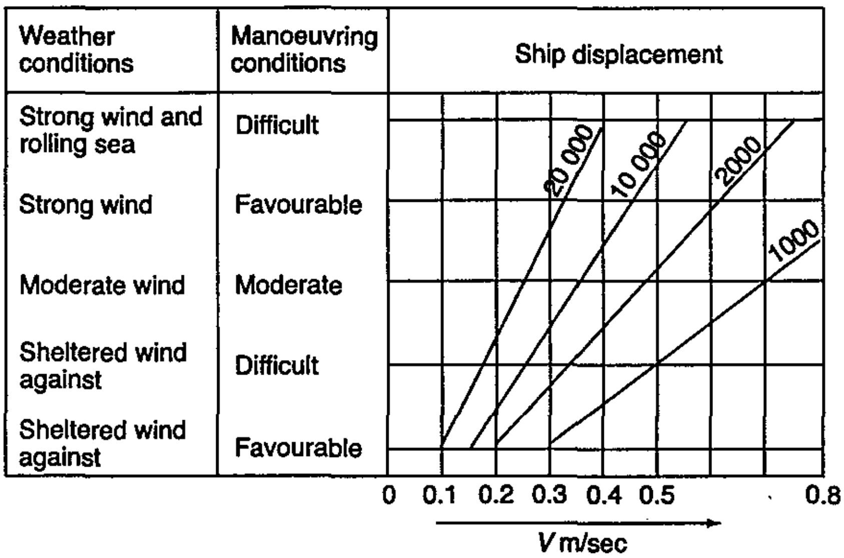

Fig. 5.3. Velocity of ship coming alongside without tugboat assistance

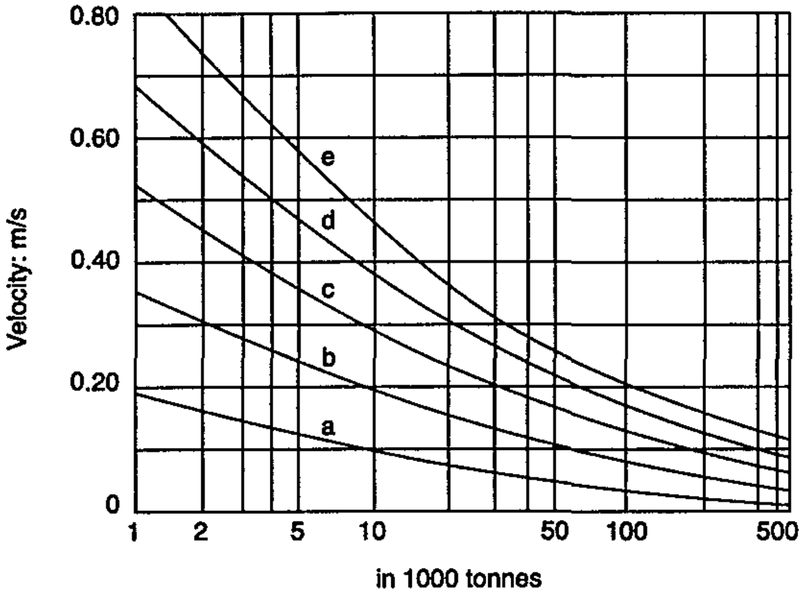

Fig. 5.4. Design berthing velocity (mean value) due to the ship displacement with tugboat assistance

systems for larger ships, the following berthing velocities **with use of tugboat** assistance should not be less than:

Very favourable conditions	10 cm/s
In most cases	15 cm/s
Very unfavourable conditions with cross current and/or much wind	25 cm/s

In 1977 Brolsma *et al.* (PIANC, 2002) recommended the berthing velocity (mean value) with tugboat assistance shown in Fig. 5.4, differentiated into the following five navigation conditions with tugboat assistance and the size of the ship in displacement:

(a) good berthing conditions, sheltered
(b) difficult berthing conditions, sheltered
(c) easy berthing conditions, exposed
(d) good berthing conditions, exposed. This figure is considered to be too high
(e) navigation conditions difficult, exposed. This figure is considered to be too high.

The Japanese section of the PIANC has collected information on the relationship between the ship's displacement and the approaching velocity for large cargo ships and tankers, as shown in Fig. 5.5. The approach of the ships was made in such a manner that the ship was

Table 5.2. The velocity in m/s and with tugboat assistance

Ship displacement in tonnes	Favourable condition	Moderate condition	Unfavourable condition
Under 10 000	0.20–0.16	0.45–0.30	0.60–0.40
10 000–50 000	0.12–0.08	0.30–0.15	0.45–0.22
50 000–100 000	0.08	0.15	0.20
Over 100 000	0.08	0.15	0.20

stopped parallel with and about 10–20 m off the berth structure and then gradually pushed to the berth structure under the full control of several tugboats.

When designing the fenders for the ramps for ferries and ro/ro ships, the berthing bow or stern velocity for these ships berthing under their own power, will, depending on the stopping distance which the

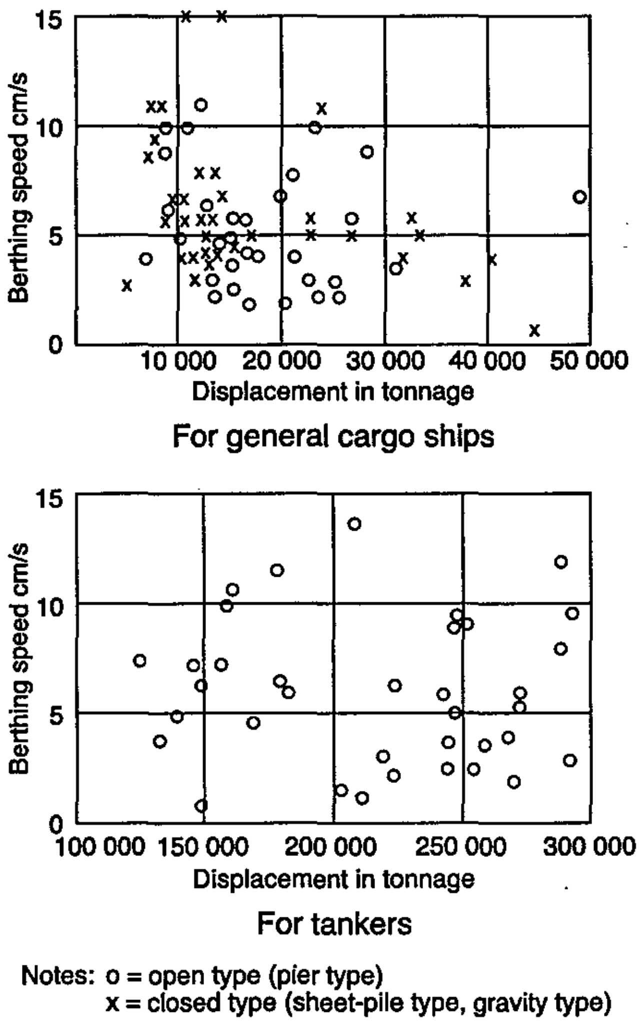

Fig. 5.5. Relation between berthing speed and ship displacement

actual ship will use in relation to the length of the berth, vary generally between about 0.4 to 1.0 m/s.

Due to the rapid turnaround times and high engine power of most ferries, the berthing velocities are generally higher than for other ships. It is therefore recommended that the berthing velocity in the direction of approach for the fender design should be:

(a) For fenders at a corner of the berth structure or the outer or end breasting dolphin: 2.0–3.0 m/s.
(b) For fenders along the berth structure: 0.5–1.0 m/s.

5.3 The empirical method

The velocity of approach as used in the theoretical method is the most significant and difficult element in the evaluation of the berthing energy imparted to the fender. Therefore the following empirical formula by Girgrah (1977) for the maximum impact energy in kN m to be absorbed by the fender based solely on a ship's displacement may be considered:

$$E_f = \frac{10M_d}{120 + \sqrt{M_d}}$$

where M_d is the displacement tonnage of the berthing ship. A factor of 0.5 may be applied in cases where the impact would be either shared between two fenders or accompanied by rotation of the ship.

5.4 The statistical method

The statistical design method is based on measurements of the impact energies actually absorbed by the fenders during berthing. As the method is based on the measurements actually observed at existing berth sites it automatically includes the effect of the berthing velocity, hydrodynamic mass, eccentricity, etc.

In Fig. 5.6 the impact energy during berthing operations in normally protected harbours is shown as a function of the displacement of the ship. One curve shows the measurements of the energy in Rotterdam. The other two curves show the impact fender energies recommended by the British Code of Practice and the Norwegian Standard for berth structures.

The Norwegian Standard also mentions that for harbours exposed to strong winds and current, or with particularly difficult manoeuvring

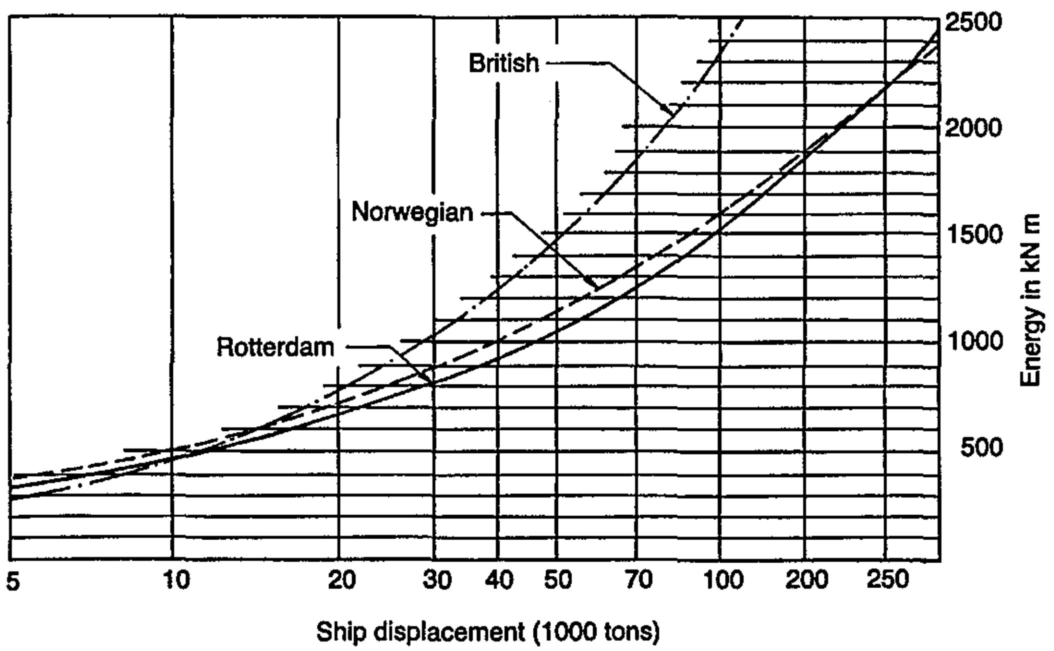

Fig. 5.6. Impact energy during ship berthing to a berth structure

conditions, the impact energy given in Fig. 5.6 shall be increased by up to 50 per cent. For structures in the open sea the impact energy shall be increased by up to 100 per cent.

5.5 Abnormal impacts

An abnormal impact occurs when the normal calculated energy to be absorbed at impact is exceeded. The fender systems have to cater for the normal impacts due to the design ship and to be capable of catering for a reasonable abnormal impact. The reasons for abnormal impacts can be bad manoeuvring, mishandling, exceptional winds or currents or a combination of them. The abnormal impact factor should enable reasonable abnormal impacts to be absorbed by the fender system without damage.

The abnormal impact factor should, according to the PIANC Fender 2002, take into account the following:

(a) Berths with high frequency of berthing will have a higher probability of abnormal impact and therefore a higher factor.
(b) The effect a failure on the fender system would have on berth operations.
(c) Berths that have been designed for very low approach velocities would be more likely to incur abnormal impact than berths designed with higher approach velocities.

Table 5.3. Fender safety factors for abnormal impact

Type of ship	Factor of abnormal impact	General comments
Small ships	1.5–2.0	Depends on operation
General cargo	1.75–2.0	Depends on ship size
Ro/ro	2.0–3.0	Stern berthing
Ferries	2.0–3.0	Depends on berth exposure and operation
Tankers and bulk	1.3–2.0	Smallest ships
	1.3–1.5	Largest ships
Container ships	2.0	Smallest ships
	1.5	Largest ships

(d) The vulnerability and importance of the berth structure supporting the fender system. If the time and costs involved in repairs are likely to be disproportionately large, a higher abnormal factor should be used.
(e) Where a wide range of ships uses the berth and the largest ships only use the berth occasionally, the factor of abnormal impact may be reduced.

Care should be taken not to increase the factor for abnormal berthing to such an extent that the fender capacity and consequently the fender reaction forces become detrimental to small ships using the berth. It should be noted that a factor of safety of 2.0 for the berthing energy provides for only a 40 per cent increase in the ship speed, as the berthing energy is related to the square of the speed.

Suggested fender safety factors for abnormal impact to be applied to the designed fender energy should be as shown in Table 5.3.

For the chain system used for suspension of the fenders and for fender panels and fender walls the normal impact factors should be between 3 and 5. For abnormal impacts when the chain loads may be higher, the abnormal factor should be at least 2. The highest factored load from either normal or abnormal impacts should be less than or equal to the minimum breaking load of the chain.

5.6 Absorption of fender forces

When the energy to be absorbed by the fender system has been established, one should select a fender which will transmit an acceptable horizontal force against the berth-structure front. This horizontal force will depend on the characteristics of the fender. It should be

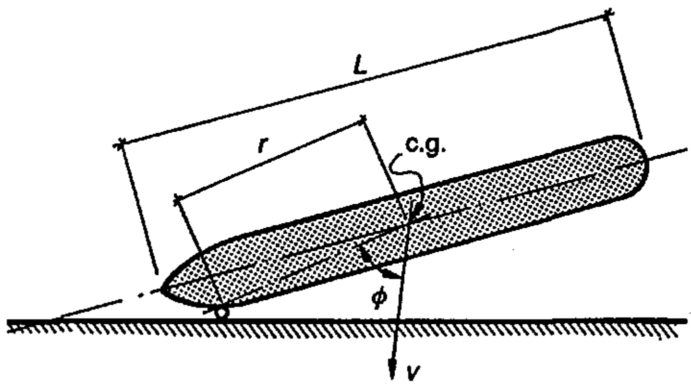

Fig. 5.7. Ship coming alongside the berth under own power

taken into account that the ship will also have to resist this force. Generally it is desirable to have these horizontal forces, or reaction force and corresponding reaction pressure, as low as possible to avoid damage to the side of the ship, and to minimize the construction cost of the berth. It is an often-discussed question how great a part of the fender will be actively resisting the impact. If the ship comes alongside under its own power, which is the most usual way, it will form an angle with the berth line, as shown in Fig. 5.7. It is, therefore, a generally accepted design practice, that each fender unit in the system should have sufficient energy-absorbing capacity to absorb the largest impact load. Each fender unit must be capable of absorbing the full impact energy or load since ships almost always contact only one fender on the first impact.

If the ship has been assisted by tugboats and is berthed parallel to the structure, or is manoeuvred parallel to the structure by help of wind or current, the length of the area of contact between ship and berth structure will still be smaller than the length of the ship, i.e. $L_{sf} < L_s$, as shown in Fig. 5.8. This fact is of great importance as regards choice of

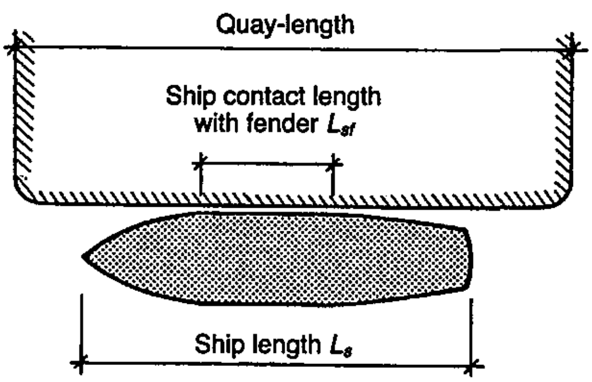

Fig. 5.8. Length of contact area L_{sf} is smaller than length of ship L_s

Table 5.4. Point loads and loads per lin metre

Displacement tons	Point load kN per 0.25 m^2	Line load kN per lin m berth
2000	100	15
5000	200	15
10 000	300	20
20 000	500	25
30 000	600	30
50 000	800	35
100 000	1000	40

fender type, spacing of the fenders and horizontal force acting on structure and ship. For instance, the length of the contact area for some container ships can be as small as about 20 per cent of the ship's length, while it can be up to 70 per cent for a traditional general cargo ship.

To ensure that the front of the berth structure has satisfactory safety under normal calls, the German *Recommendations of the Committee for Waterfront Structures* (EAU, 1996) recommends that the berth front is designed for a horizontal point load equal to the bollard load. This point load shall be allowed to act anywhere at the berth front without the allowable stresses being exceeded, and its contact area shall be limited to 0.25 m^2. Table 5.4 gives point loads and the corresponding increased loadings in kN per lin m of, for example, a berth for various ship displacements.

When a ship comes alongside at an angle with the berth line, longitudinal friction forces parallel to the berthing face will be transmitted by way of the fenders to the berth structure. When this occurs the ship will skid along the structure after the impact while the fenders are still in a compressed state. The front of the berth structure must take up this friction force $F = \mu \times P$, where μ is the friction coefficient between ship and fender, and P is the impact force.

The friction coefficients μ are for steel-to-steel 0.25, for steel to polyethylene 0.2, for timber to steel 0.4 to 0.6 and for rubber to steel 0.6 to 0.7. The friction force F is usually acting simultaneously with the impactive force perpendicular to the front of the structure. The longitudinal friction forces can be reduced by provision of low-friction contact surface materials.

The horizontal forces due to long periodic wave action along the berth front are dependent on the length of the long periodic wave in relation to the size of the moored ship. On a wave slope of 1 in 2000,

Table 5.5. Up-and-down directed loads

Ship displacement in tons up to	Vertical up/down directed load in kN/m of berth
2000	10
5000	15
10 000	20
20 000	20
30 000	25
50 000	25
100 000	30

the berth parallel forces for a ship of 300 000 tons displacement are about 1500 kN. With a friction coefficient between the fenders and the ship's hull of 0.7, the fenders and the berth structure must take up a horizontal force along the berth front of about $1500 \times 0.7 = 1050$ kN. If the ship is moored by forced or tension mooring to reduce the surge movement, the mooring forces normal to the berth front will be $1500/0.7 = 2150$ kN which is equivalent to four winches of about 550 kN capacity.

5.7 Ship 'Hanging' on the fenders

When a ship is moored at a berth structure it can 'hang' itself on the fenders due to tidal variations, or friction between the ship's hull and the fender, or it can chafe on the fenders during loading and unloading. The front of the structure should therefore, in cases where such effects are possible, be designed for up-and-down directed loads as suggested in Table 5.5.

Where big, energy-absorbing rubber fenders or similar protruding fenders are used, such vertical loads must be estimated in each case.

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6

Design considerations

6.1 General

Normal loading refers to any combination of loads that may reasonably be expected to occur during the structure's design life and under normal operating conditions. Extreme loadings are any loading combinations that may be expected to occur during the design life of the structure, included the most severe physical loading that could be applied, excluding accidental loads such as uncontrolled berthing.

Conditions, described as limit states, are defined states which could occur during the design life of the structure so that it would fail to fulfil satisfactorily its intended functions or that it would become unfit to do so. Sets of factors are then specified so that the probability of each limit state occurring during the design life of the structure does not exceed a value agreed to be acceptable having regard to the consequences of the limit state occurring.

The design load for a limit state is defined as the most unfavourable combination of the characteristic load multiplied by a load coefficient. The limit states are categorized as follows:

(a) **The ultimate limit state** (ULS) is related to the risk of failure or large inelastic displacements or strains of a failure character.
(b) **The serviceability limit state** (SLS) is related to criteria governing normal use or durability.
(c) **The fatigue limit state** (FLS) is related to the risk of failure due to the effect of repeated loading.
(d) **The progressive collapse limit state** (PLS) is related to the risk of failure of the structure under the assumption that certain parts of the structure have ceased to perform their load-carrying functions.

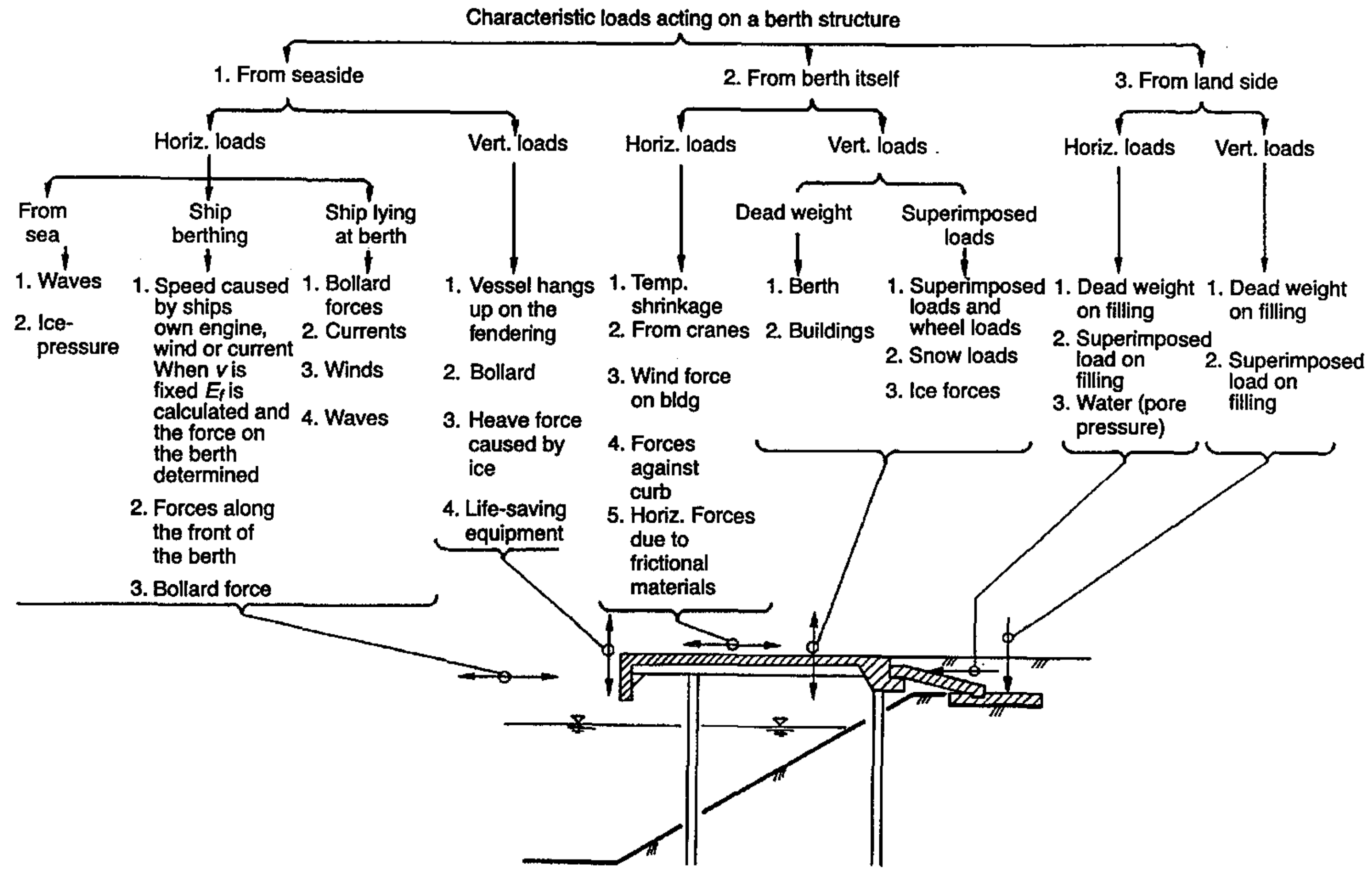

Fig. 6.1. Characteristic loads acting on a berth structure

In the different limit states, there are three main categories of characteristic loads or forces acting on a berth structure:

(a) characteristic loads from the sea side
(b) characteristic loads on the berth structure
(c) characteristic loads from the land side.

The characteristic load is defined as the load which has a known probability *p*, based on annual extremes, which it will not be exceeded in an individual year. The characteristic load may be a permanent load, variable load of a return period of 50 years, fatigue load or accidental load.

According to the acceleration that the load gives to the structure it is also divided into **static load** and **dynamic load**. The load coefficient does not include the dynamic allowances.

The objectives of the characteristic load criteria given in this chapter are to ensure that structures and structural elements among other things are designed to:

(a) sustain all loads and deformations with an acceptable degree of safety against failure
(b) perform adequately in normal use with respect to deterioration, displacements, etc.
(c) have adequate fatigue resistance.

Figure 6.1 shows in some detail the various types of force which usually occur on a berth structure. In this chapter the characteristic loads acting on the berth structure itself and the loads from the landside will be discussed in more detail. The characteristic loads from the seaside are mainly given in Chapters 2 and 5.

The Norwegian design practice for open berth structures recommends that, if possible, all vertical loads on the berth structure are taken by the piles or the columns, and that all horizontal loads are taken by the friction slab behind.

6.2 Design life

The estimated design life of a berth structure, which is taken to be equal to the useful life of the structure with planned maintenance, is a major concern that can greatly influence the structural design. As a rule of thumb one can say that for ordinary berth structures in commercial ports the design life should be at least 50 years. For berths serving special industries, container traffic, oil traffic, etc., a period of not

Fig. 6.1. Characteristic loads acting on a berth structure

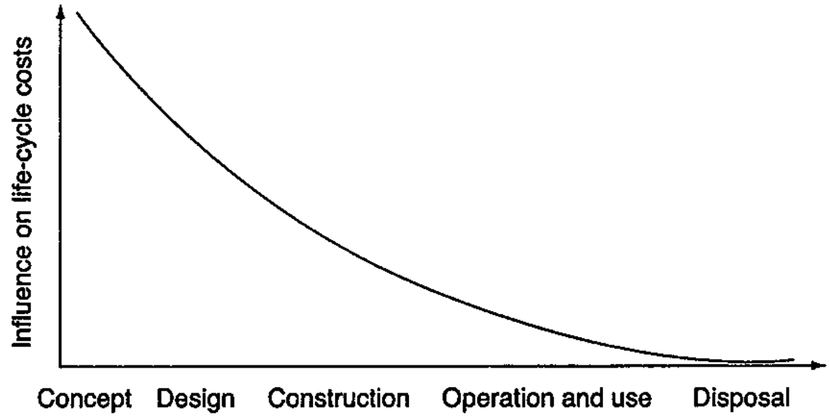

Fig. 6.2. Influence of the main phases in asset life upon total life-cycle costs

more than 30 years is often more relevant. The background for these figures is that modern specialized cargo handling is more subject to rapid development, which may also lead to a relatively early outdating of the berth facilities (e.g. the influence of the container-handling technology on the berth structure). For shore protection works and breakwaters a design life of 100 years, and for flood protection works a design life of more than100 years, will normally be appropriate.

There is a well-known joke that the design life of a structure should at least be as long as the remaining engineering career of the designer.

The total design and service life of a berth structure or any type of structure would depend on all the following factors:

(a) The design engineer's experiences in the design work.
(b) The design standards and recommendations used.
(c) The correct materials used for the environment in which the structure is situated.
(d) The construction of the berth structure, and the contractor's ability to do the job properly.
(e) The proper inspection of the berth structures, e.g. every third year, and maintenance and repair of any damage.

Figure 6.2 indicates the fact that the influence on the total life-cycle costs is progressively reduced from concept and design, construction to operation and use to disposal of the structure.

The determination of design life should consider the following:

(a) Assessment of the factors, that influence the security of the structure. These may include fatigue loading, corrosion, marine growth and reduction in soil strength. Therefore the acceptable probability of failure or the acceptable degree of damage during the structure's lifetime should be decided at an early stage of the design.

(b) Evaluation of the probabilities that particular limit states will occur during the design life.
(c) Appraisal of economic feasibility, necessity to allow for developments and related matters. Therefore the costs of, say, repair work should be estimated and included in the evaluation of the economic feasibility of the project.

For instance, a berth slab consisting of slender prefabricated prestressed concrete elements may represent an economically good solution if the useful life of the berth is estimated to be only approximately 10–15 years. Based on experience with salt penetration and corrosion in concrete structures in marine environments, such types of slab should not be chosen for berths that are supposed to have a long useful life of more than 50 years due to the fact that it is practically impossible to repair prestressed concrete elements.

In view of the essentially variable and often unpredictable character of the loads to which maritime structures are subjected, it is unrealistic to expect any substantial cost savings to result from attempting to design them for short lives. Generally there will be greater economy to be achieved in aiming at simplicity and robustness of the overall concept and construction methods.

The average length of the economic life for port structures and port equipment will depend on the degree of maintenance, and of the period and thoroughness of and cost spent on the maintenance. As a very rough guideline Table 6.1 shows very average values can be used as an indicator.

Table 6.1. Average maintenance costs

Part of the port structure and type of equipment	Average economical design life in years	Annual average maintenance costs as a percentage of new costs or replacement value
Breakwater	100	2
Reinforced open berth structure	50	1–2
Steel sheet-pile berth structure	50	1–2
Rubber fenders	10	1
Concrete aprons and roads	20	1–2
Asphalt surfacing	10	2
Container gantry cranes	20	5
Mobile container cranes	15	10
Fork-lift and reach stackers	10	10
Straddle carriers	5–10	10–15
Road tractors	10	10
Warehouses and sheds	40	5

The most important feature of maintenances costs is regular inspections and reporting, upon which a routine maintenance system can be developed as a guide in order to keep the structure in good repair and have a long service life rather than be allowed to deteriorate over a period of time.

6.3 Load factors

The structural design calculations are generally performed according to the partial safety factor method by applying partial safety factors on both the action side and the resistance side. It is imperative when performing design that consistent sets of standards are used where design formulas, load and material factors are all calibrated to give the intended reliability.

When designing structures these are checked for a large number of possible combinations of loads. Typically one uses a representative or characteristic value for actions as the basis, and adjusts these to an acceptably low probability of the action effects being exceeded by applying a set of factors accounting for both variations and deviations in action values, model uncertainties and dimensional variations.

When more than one variable action is included in the load combination it will often be relevant to consider one as the leading variable action and those remaining as accompanying variable actions, where the accompanying variable actions have a combination value which is less than the characteristic value. The appropriate combination value will be obtained by a factor (ψ), which is dependent on whether the situation that shall be represented is a normal ULS condition, a frequent value or a quasi-permanent value.

When using, for example, the Norwegian design standard for concrete structures to design, say, berth structures one should also use action factors according to the Norwegian reliability standard. The applicable action factors would then be 1.2 for permanent loads, 1.5 for the leading variable action and not less than 1.05 for the accompanying variable actions.

In Europe, the *Eurocodes* are taking over as the basic standards for design. Matters such as reliability and durability are, however, within the competence of the various member states. This is provided for by Nationally Determined Parameters (NDPs) on parameters influencing reliability, etc., and are given in National Annexes to the standards. The effect of this will consequently be that, although there is a common set of design standards, namely the *Eurocodes*, a number of parameters will vary between the different countries. It is consequently therefore important to use the set of NDPs applicable in the specific country where the harbour and/or berth structure are to be built.

The action factors, the combination factors and the equations for how the actions shall be combined are given in *Eurocode EN 1990*. By using the *Eurocode* one will give a more consistent and uniform reliability independent of the reaction of permanent loads and variable loads, and it could therefore be recommended to use this combination, if permitted by the National Annex (NA).

The combinations of actions are given by two sets of equations often referred to as either equation 6.10 or equations 6.10a and 6.10b in combination. Using equations 6.10a and 6.10b will give a more consistent and uniform reliability independent of the ratio of permanent loads and variable loads. It is therefore recommended to use this combination, if permitted by the NA.

$$\sum \gamma_{G,j} G_{k,j} \text{ “+” } \gamma_p P \text{ “+” } \gamma_{Q,1} Q_{k,1} \text{ “+” } \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{6.10}$$

$$\sum \gamma_{G,j} G_{k,j} \text{ “+” } \gamma_p P \text{ “+” } \gamma_{Q,1i} \psi_{0,1} Q_{k,1} \text{ “+” } \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{6.10a}$$

$$\sum \xi \gamma_{G,j} G_{k,j} \text{ “+” } \gamma_p P \text{ “+” } \gamma_{Q,1i} Q_{k,1} \text{ “+” } \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{6.10b}$$

The *Eurocode* does not give recommended values for action factors and combination factors directly applicable for harbour and berth structures. The magnitudes of the factors have to reflect the probability of loads being exceeded. For example, the combination value on actions in open areas used for traffic should be less than in areas for storage, and higher if the stored cargoes typically have a weight close to the characteristic value than if the weight is a random variable and normally well below the characteristic value. In lieu of other values the following could however be considered:

Equation 6.10a:
- permanent action, action factor γ_G; 1.35/1.0 (when unfavourable)
- accompanying variable actions (main and other variable actions), combination and action factor $\gamma_{Q,i}\psi_{0,i}$; $1.5 \times \psi_{0,i}$
- combination factor $\psi_{0,i}$ should be taken in the range 0.8–1.0.

Equation 6.10b:
- permanent action, reduced action factor $\xi\gamma_G$; $\xi \times 1.35$ or if unfavourable $\xi \times 1.0$
- ξ should be taken in the range 0.85–0.9
- leading variable action, action factor $\gamma_{Q,1}$; 1.5

– other variable actions, combination and action factor $\gamma_{Q,i}\psi_{0,i}$; $1.5 \times \psi_{0,i}$
– combination factor $\psi_{0,i}$ should be taken in the range 0.8–1.0.

6.4 Material factors

The design strength of a material is determined by dividing the structural material strength by a material factor, which shall account for uncertainties in material strength, execution and calculations. The material factor must also take into account the consequences of damage. The consequences of damage can be divided into three classes:

(a) Less serious: failure that involves little risk of injury to people or small economic or other consequences.
(b) Serious: failure that involves risk of injury to people or significant economic or other consequences.
(c) Very serious: failure that involves large risk of injury to people or very large economic or other consequences.

The values of the material factors are designed to provide a level of safety appropriate to the purpose of maritime structures. In the maritime environment, considerations of damage to material objects rather than to human life generally predominate. It is therefore necessary for a rational design to weigh the ascertainable cost of providing additional strength against the probable costs of repair, consequential damage and economic loss during the life of the structure.

In, for example, the Norwegian Standard, the material factors for concrete and reinforcement are 1.4 and 1.25 respectively for concrete works executed under an extended or ordinary inspection work standard. The standards allow the use of reduced material factors in cases where the tolerances are strictly controlled and where the maximum deviations in the most unfavourable direction are considered in the design. For berth and harbour structures where the durability is a major concern, the use of such reduced values is however not encouraged. On the basis of experience with maintenance and deterioration of structures in marine environments it is recommended that a material factor at least equal to the following, if the design life of the structure is more than about 20 years, should be used:

(a) steel piles filled with reinforced concrete: 1.25
(b) reinforced concreted piles, etc. concreted under water: 1.60
(c) all other harbour structures: 1.40.

If the design life of the structure is less than 20 years a material factor of 1.25 can be used.

In the new European suite of design standards, the *Eurocode*, the material factors are defined in a slightly different manner. The *Eurocode* is a common set of standards, although matters related to both safety and durability are within the competence of the member states. Parameters that affect safety and durability are therefore open for the member states to determine as NDPs. The *Eurocode* gives recommended values, but the various European countries are free to give other values that have to be used on their territory. These NDPs shall be given in a NA. The recommended values for reinforced concrete structures are for the concrete $\gamma_c = 1.5$ and for the reinforcement $\gamma_s = 1.15$. Reduced values may be permitted under certain conditions.

The design strength of the concrete is determined as $f_{cd} = \alpha_{cc} f_{ck}/\gamma_c$ where α_{cc} is a factor taking account of long-term effects on the compressive strength of the concrete resulting from the duration of the load and the way it is applied. The *Eurocode* specifies that α_{cc} shall be taken between 0.8 and 1.0 with 1.0 as the recommended value. For berth structures it may well be that a value of 0.9 should be considered for α_{cc}.

6.5 Characteristic loads on berth structures

6.5.1 Temperature and shrinkage forces

In the design of, for example, berth decks of reinforced concrete, allowances must be made for temperature and shrinkage forces in the transverse as well as the longitudinal directions of the deck.

6.5.2 Live loads and wheel loads

It is difficult to lay down guidelines for live loads on aprons as a function of the ship's size. The loads on the apron deck are determined by the type of traffic utilizing the berth, and not so much by the size of the ships. Special berths like oil piers accommodate ships of several hundred thousands of tons displacement but have live loadings of say 10 kN/m^2. On the other hand, berths accommodating supply ships for the offshore oil industry of only, say, two thousand tons displacement must be designed for a live load of between 50–200 kN/m^2. Berths for heavy industry should be designed for a live load of between 40–100 kN/m^2. In fishing harbours the berth structures should be designed for a live load of at least 15 kN/m^2. The loads are therefore essentially dependent on the type of cargo, on the

Table 6.2. Recommended live loads

Type of traffic and cargo	Loading in kN/m^2
Light traffic or small cars	5
Heavy traffic or trucks	10
General cargo	20
Palletized general cargo	20–30
Multi-purpose facility	50
Offshore feeder bases	50–200
Heavy vehicles, heavy crane, crawler crane, etc. that operate from the berth front and 3 m inboard	60
Heavy vehicles, heavy crane, crawler crane, etc. that operate from 3 m behind the berth front and further inwards	40–100
Containers	
Empty and stacked 4 high	15
Full and stacked 2 high	35
Full and stacked 4 high	55
General ro/ro loads	30–50

handling equipment, local practices, etc., so that uniformity can only be achieved to a limited extent.

As a general guideline Table 6.2 shows recommended live loads for the apron and the terminal area.

In the case of a very exposed open berth structure, the possibility of uplifting of the deck structure due to waves passing under should be considered.

It is strongly recommended that the same live load for the whole terminal as the live load used at the apron be used, in order to achieve maximum flexibility in cargo-handling techniques. The berth structure should also be designed to carry the maximum live loading that might be imposed during the life of the structure due to handling, transport and storage of the cargo or other activities.

Most public berths (multi-purpose berths), accommodating ocean-going dry-cargo ships, should be designed for container loads. Twenty-foot containers stacked two high imply a load of 25–35 kN/m^2 depending on the cargo they are loaded with. The sizes of a 20-ft and 40-ft container is respectively $6.06 \times 2.44 \times 2.44$ m and $12.12 \times 2.44 \times 2.44$ m. The empty weight of a 20-ft container range between 19–22 kN and the maximum total weight permitted by ISO (the international container standard) is 240 kN. For a 40-ft container the empty weight ranges between 28–36 kN and with a maximum total weight of 305 kN.

Aprons and ramps for container traffic should be designed for a useful load of at least 40 kN/m^2.

Wheel loads from trailers, fork-lift trucks, mobile cranes, container cranes and other cranes on rails, railways, etc. should be evaluated in each case, because there is, in the market nowadays, a spectrum of types and makes of such equipment with individual loading specifications. Fork-lift trucks for handling 40-ft containers can have axle loads of up to 1200 kN. In order to highlight the relatively big damaging effect of fork-lifts on pavements, it is significant to note that an axial load up to 1200 kN on a fork-lift will give a wheel load slightly higher than the maximum wheel loads transmitted to the pavement during take-off by a Boeing 747 B.

Where mobile cranes may operate in the area behind the berth line, then provision should be made for the outrigger reactions and bearing pressures which may be imposed by the maximum size of a crane anticipated. The outrigger reactions are largely dependent on the crane lifting capacity and the radius of the jib length. If no information on the mobile crane can be obtained, the berth structure or the apron should be designed for a concentrated point load of at least 700 kN on an area of 1.0 m × 1.0 m in the least favourable position. It should be mentioned that wheel loads for railways would be increased by 10 per cent and for fork-lift trucks and cranes by 20 per cent due to dynamic impacts. Both the berth apron and the whole container yard have to be designed in a homogeneous way and must be able to carry the heaviest combination of wheel or static loads for all handling equipment that may be present in the areas, i.e. container cranes, trailers, fork-lifts, straddle carriers, etc. In Chapter 13 about container terminal equipment, the different types of container handling equipments are in shown in principle.

To reduce the effect of a concentrated point load acting directly on a concrete deck slab, or to increase the loading area on which a concentrated point load is acting, one can, as shown in Fig. 6.3, put a layer of sand and asphalt on top of the concrete slab.

Berth structures which have direct road connection to the public highway network, should at least be designed for loads in accordance with the Highway Department's regulations. The loads should be at least a concentrated load of 150 kN on an area of 0.2 m × 0.2 m in the most unfavourable position, or a live load of 20 kN/m^2.

The horizontal load transmitted to the apron, due to braking or wind forces, from cranes is about $\frac{1}{7}$ of the wheel load on the braked wheels in the direction of the rails. The horizontal load in the direction

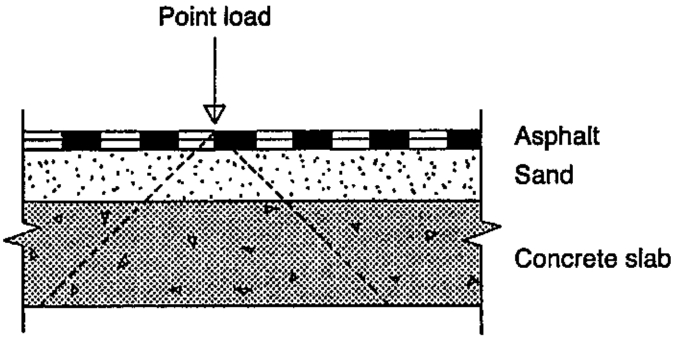

Fig. 6.3. To increase the loading area

perpendicular to the rails is about $\frac{1}{10}$ of the wheel load. For rubber-tyre mounted equipment a factor of $\frac{1}{10}$ is also applied.

Storing of frictional material will imply a horizontal load component being transmitted to the apron deck in addition to the vertical load. This component, acting as a tensile force in the top of the deck, is equal to the maximum static friction in the stored material.

In order to prevent vehicles from rolling over the berth line into the water, a front curb should be installed along the berth front. This curb should be designed for a horizontal point load of 15–25 kN depending on the type of traffic and should be about 0.20 m high.

Useful loads in transit sheds and warehouses depend to a great extent on the height to which palletized cargo can be stacked with fork-lift trucks. Design loads vary between 20 and 50 kN/m^2 (or more) over the whole floor area, depending on types of cargo.

To prevent overloading of the berth structure the allowable load should be marked in clear letters and figures on a signboard at the apron.

6.5.3 Seismic loads

Seismic or earthquake loads on the berth structure should be considered if the structures are in an area of seismographic disturbance. The seismic loads will act at the centre of gravity of the structure as a horizontal force equal to the design coefficient times the weight of the structure. The weight to be used for the berth structure itself should be the total dead load plus one-half of the live load. The design seismic coefficient is equal to the regional seismic coefficient times the factor for the subsoil condition times the coefficient of importance. The design seismic coefficient will usually be between 0.05–0.25. For cargo-handling equipment, the seismic load is the product of a horizontal seismic coefficient and the deadweight of the cargo-handling equipment.

The actual seismic load due to an earthquake will depend on the magnitude of the earthquake, the type of structure, type of equipment and the soil conditions in the area. Generally, unless the berth structure is of a massive or gravity type, the seismic effect on the design will usually be small. This applies to both the transverse and the longitudinal direction of the berth structure.

The seismic performance requirements for a particular berth structure should be established in accordance with international standards and guidelines based on acceptable risk procedures. The requirements for a berth structure should be based on the importance of the berth structure, the acceptable levels of risk to life safety, the port operations, etc.

6.6 Characteristic loads from the landside

The weight of the fill behind the berth structure and the useful load on top of the fill will serve as a stabilizing load, for example on berth anchoring friction plates. The weight of the fill may also cause horizontal loading to the berth structure, for instance in connection with water pressure due to a blockage of the drainage system behind the berth structure. The magnitude of such forces must be evaluated in each separate case.

A further discussion of forces acting from the landside is considered to be outside the scope of this book, but EAU 1996 and ROM 0.2-90 give useful recommendations.

6.7 Summary of loads acting from the seaside

From the above discussions not only are static and dynamic conditions involved when design loads on the berth structure being established, the human factor during, for example, manoeuvring the ship to the berth also comes in. Therefore this suggests the assumption of more conservative load values than those which are strictly necessary according to detailed calculations.

As an example, if the contractors carrying out a tender are allowed to give alternative designs for example, berthing structure for gas tankers, the following minimum design loads should be given.

Ship size maximum	137 000 m^3
Ship size maximum fully loaded displacement	100 000 t
Ship size minimum	20 000 m^3

Approach ship velocity normal to jetty front	0.15 m/sec
Approach ship velocity parallel to jetty front	0.02 m/sec
Ship angle of approach to jetty front fully loaded	5°
Hull pressure between the fenders and the ship, max.	0.20 MPa
Friction coefficient between ship hull and fender, both horizontal and vertical	0.2
QRH with capstan, minimum capacity of each hook	1000 kN
Loading platform and access road:	
Point load at any point at loading platform and access road on area 1 m × 1 m	700 kN
Vertical live load, general	20 kN/m^2
Pipeline:	
Vertical live load	2.5 kN/m^2
Walkways:	
Vertical live load	3.0 kN/m^2
Horizontal load top handrail	0.8 kN/m
Mooring dolphins:	
Vertical live load	3.0 kN/m^2

Earthquake: There is little risk of seismic activity. For design purpose the values of z according to Uniform Building Code (USUBC) are assumed to be as for zone 2B, where $z = 0.2$.

References and further reading

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7

Safety consideration

7.1 General

The safety measures, which have to be considered in a harbour project, will be safety related to the specification, the design, the construction, the personnel and the operation. The safety related activities are important activities in the work of the Consulting Engineer and all these aspects should therefore be given the highest priority during all his consulting work.

7.2 Specification safety

For a harbour project, the primary activity of securing satisfactory implementation of safety should be considered in the specification or the start-up phase. In this phase all the engineering standards, design codes, governmental laws and regulations have to be defined and listed as the project engineering specifications.

The safety routines for all the work to be performed by the Consulting Engineer should be implemented through quality assurance and the control system for the project, and through the project coordination and engineering procedures. The project quality assurance manual, project coordination and engineering procedures should give detailed regulations for review and approvals, both internal for the project team, and in relation to Client and external interfaces.

7.3 Design safety

The design aspect for the berth structures should be based on common and proven design methods and technology. In order to sort out the different load conditions, which may occur during the lifetime of a berth structure, it is convenient to distinguish between the following three conditions:

Operational condition: the design condition, which takes into account the normal design loads based on the national standards or recommendations for berthing structures. In this case, for example, the fender system should be able to absorb the normal design berthing energy related to the approach velocity without damaging either the berth structure or the ship itself. Due to safety reasons, the maximum acceptable approach or berthing velocity should not be more than about $\frac{2}{3}$ of the design approach velocity for the berth structure. The design berthing energy should not be higher than the recommended absorbed or rated energy of the fender by the fender manufacturer.

Accidental condition: for example, the 'engines out' condition that may happen to a berthing or unberthing ship. In this case the berthing energy may be higher than in the operational condition. Damage to the fenders may be allowed or expected to occur in this condition, but the concrete breasting structure itself should be constructed so as not to collapse under such an impact. In accidental conditions the concrete structure should be able to resist a horizontal force due to a 20–25 per cent higher berthing energy than the design berthing energy in the operational condition without a total collapse of the berth structure. See Section 5.5 about abnormal impacts. This is because it is very difficult to define limiting values for the exact value of a ship's approach velocity. The velocity will depend upon the wind, waves, current and the number of tugboats assisting during the berthing operation.

The more difficult it is to estimate the characteristic loads against a structure, or if the consequences of a collapse of a structure will be very serious, the more important it is that the structure has as high a reserve capacity against collapse as possible.

Catastrophic condition: this condition covers the situation where a large strange ship (e.g. a large cargo, tourist ship, etc.) impacts, for example, an oil berth construction at speed possibly causing total collapse of the structure. One has to realize that it is uneconomic and often impossible to construct a berth structure to resist such an impact. Decreasing the probability that such accidents may happen by changing the sailing routes or imposing restrictions on other ship traffic is often the only course of action.

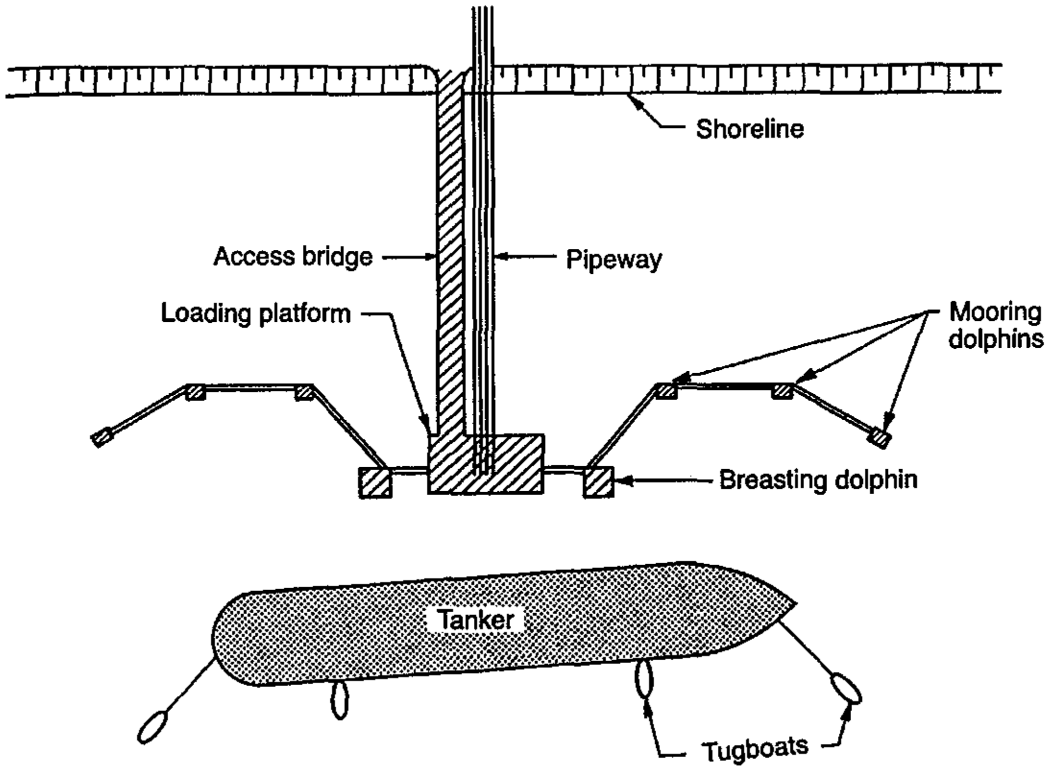

Fig. 7.1. Layout of oil jetty

The layout of an oil jetty, as shown in Fig. 7.1, is based upon the principle that the loading arms and other items of functional equipment are placed on a separate **loading platform** free from normal horizontal berthing forces from the oil tankers. The horizontal berthing or impact forces, normal to the berthing face, are taken by separate breasting structures, or breasting dolphins, as shown. This design philosophy of separating the breasting dolphins from the loading platform, reduces the possibility of damage during the berthing operation to the loading platform from normal operational or accidental berthing forces, as shown in the figure.

Different construction solutions for the cross-sections through the **breasting dolphins** in Fig. 7.1 are shown in Fig. 7.2. The breasting dolphins founded on raking piles with prestressed rock anchors, as shown in Case A, do not have any internal stability due to deadweight and make use of rock anchors to take the horizontal forces from a berthing tanker compared with a large reinforced concrete caisson filled with sand or rock fill, as that shown in Case B. If a breasting dolphin founded on raking piles with rock anchors is overloaded, due to an accidental condition, it may collapse beyond repair, or may be unusable for a long time while under repair. On the other hand, if the caisson construction is overloaded, the caisson should be so designed

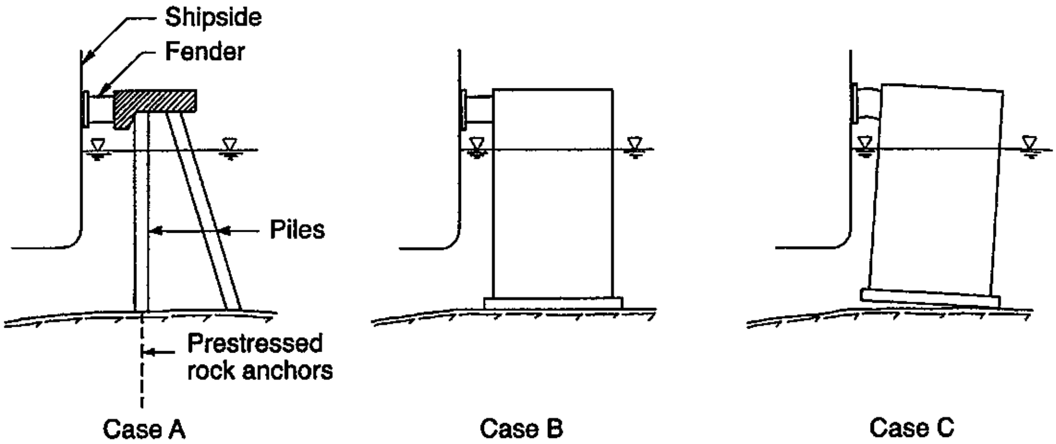

Fig. 7.2. Different breasting dolphins

that the overloading will only tilt it as shown in Case C, or push it a certain distance along the sea bottom. The repair work on the caisson could, therefore, only be a question of adding some extra material to re-establish the original fender line. The interruption of the operation of the terminal due to repair work will only be for a very short period.

If a breasting dolphin founded on raking piles with prestressed rock anchors has to be used, an additional increase in the safety can be achieved by constructing either an energy absorbing concrete or steel overloading collapsible unit between the fender and the dolphin head, as shown in Fig. 7.3, or designing the breasting dolphin itself for a reaction force 2–3 times higher than the fender reaction forces depending on the safety berthing philosophy. If few tugboats are used and the weather condition at the site is generally rough, a higher reaction force should be used in the design of the breasting dolphin itself.

The design philosophy for **mooring dolphins** shown in Fig. 7.1, assumes that the safety against overloading from the mooring hawsers can be taken care of by the anchor bolts for the mooring equipment in the concrete top slab of the mooring dolphin. The anchor bolts are only designed for a maximum horizontal force, which will not affect the stability of the mooring dolphin itself. As shown in Fig. 7.1 there are three mooring dolphins on each side of the loading platform. If an accidental condition should destroy one of the mooring dolphins, the other mooring dolphins can, in an emergency situation, be used alone while the destroyed dolphin is being repaired.

When the sea bottom is sloping steeply under the berth structure up to the water level so that the distance from the berth line to shoreline is small, as shown in Fig. 7.4, the tanker has to squeeze aside the water between the tanker and the shoreline during the berthing operation.

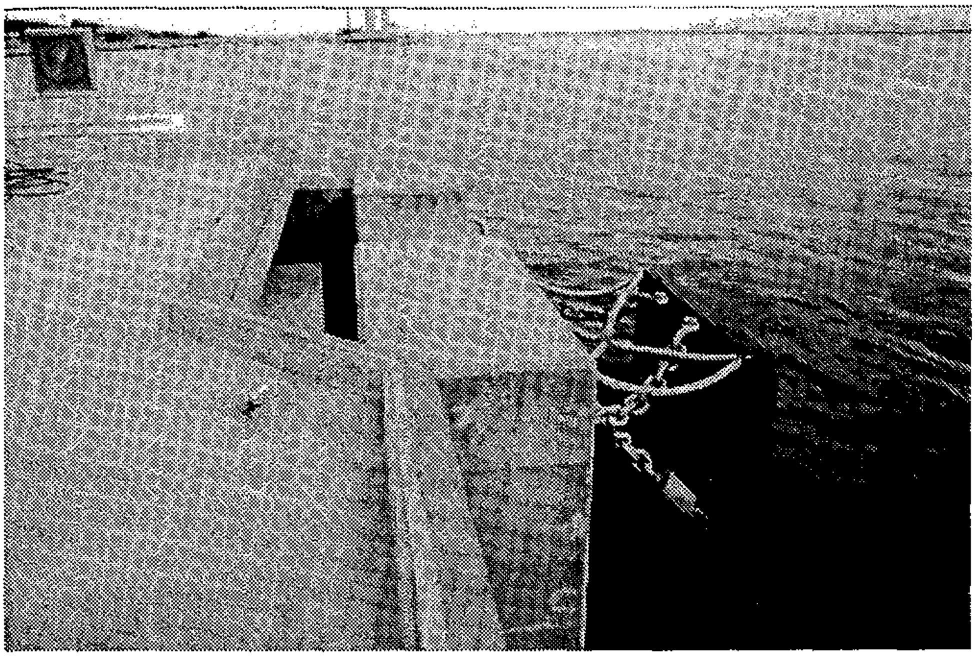

Fig. 7.3. Concrete overloading collapsible unit

This squeezing effect will have an increase in the stopping effect to the tanker compared to a totally open berth structure, as shown in Fig. 7.1. This effect is called the water cushion effect and can reduce the berthing energy by up to about 10–20 per cent. To get this water cushion effect, the shoreline must be parallel to the berth line and the parallel length of the shoreline must be at least as long as the length of the berthing tanker. To get the maximum reduction on the berthing energy due to the water cushion, the distance between

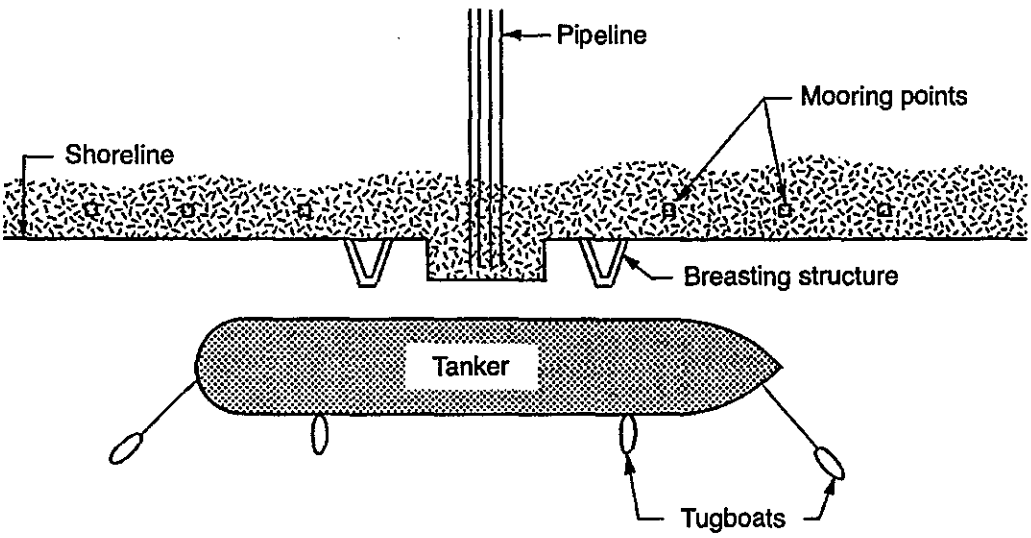

Fig. 7.4. Berthing to an oil quay

the shoreline and the berth line should not be more than about 2 m. When the distance between the berth line and shoreline is about 20 m, as shown in Fig. 7.4, the reduction in the berthing energy is only about 5 per cent after the experiences of the ship captains.

In the case of **oil spill** or pollution from a moored tanker in front of a berth structure, as shown in Fig. 7.4, it will be easier to control the pollution for this type of structure compared with oil pollution around a totally open jetty structure, as shown in Fig. 7.1. In the case of oil pollution and/ or fire, a structure as that shown in Fig. 7.4 will also have easier and safer escape routes for personnel and vehicular access.

7.4 Construction safety

Safety considerations due to construction of berth structures should, if possible, always be based on common and proven construction methods and technology. The construction solutions should be easy to construct and build, and the structure should be adapted to the construction equipment that is available.

In harbour work there should be no room for sophisticated solutions. Experiences have often shown that maintenance of harbour structures will later generally be proportional with the degree of the selection of sophisticated solutions.

7.5 Personnel safety

Safety considerations due to the personnel aspects of, for example, petroleum jetties are characterized by the following:

(a) Designed for maximum fire prevention. This means that piping should be routed and designed to avoid failure from predictable causes. Known sources of ignition should be shielded. Effective control against ignition from static current.
(b) Provided with effective fire protection. Valves should be available to quickly stop the flow of hydrocarbons through piping that is susceptible to leakage. The fire-fighting facilities should be simple to operate and easily maintained. All fire-fighting equipment should be focused on piping elements that are susceptible to leakage or vulnerable to damage, etc.
(c) Provide the jetties with effective emergency evacuation routes. Emergency and escape routes for personnel should be clearly marked. For the personnel's safety the routes should be constructed

so that it should not be necessary to jump into the water for rapid evacuation. Therefore, in case of emergency evacuation from the jetty platform, the design of escape routes must have been taken into account for the overall design of the jetty, e.g. all outer mooring dolphins should be designed with facilities for the mooring of lifeboats.

In addition one should, as far as the operations of the jetties permit, provide handrails in all places where there is a risk of falling into the sea, and safety ladders should be designed on all jetty platforms, dolphins, etc. All rescue and access ladders, bollards, curbs, walkways between mooring dolphins, etc. should be painted in orange or red self-illuminating colour.

7.6 Operational safety

For operations related to the environmental conditions during berthing and unberthing, it is very important that information on tide, current, wind, waves, etc., which plays an essential role together with the hydrographical and topographical conditions, has been collected. These factors are essential both for the safety of the ship during navigation and berthing operations as well as for the cargo-handling operations, and should be carefully considered in the planning and evaluation of proposed harbour sites and layout of berths, breakwaters and other structures.

7.7 Total safety

The total safety of a harbour will depend on the combined safety from the specification, design, construction, personnel and operational safety. To increase the total safety one has to decrease the risk of accidents occurring. The risk is equal to the probability times the consequence. To reduce the risk one has to reduce the probability and/or the consequences of an accident.

The berthing manoeuvre of an especially larger ship is always a slightly hazardous operation, and the berthing structures may be damaged resulting in consequential losses in the operation of the facility or the terminal. Therefore, these events deserve the attention of the designer as their probability and consequences may increase the total risk of the operation. Figure 7.5 shows the relative highest possibility of an accident occurring during each phase of the total navigation

	Possibility for accident to		
	Berth structure	Personnel	Environmental
Arriving outer-harbour basin			● ● ●
Turning in harbour basin	●		● ● ●
Berthing operation	● ● ●	● ●	● ● ●
Mooring	● ● ●	●	●
Loading/unloading	●	● ● ●	● ●
At berth due to bad weather	● ● ●	●	● ●
Deberthing operation	● ●		● ● ●
Departure from harbour			● ● ●

● = small possibility ● ● ● = high possibility

Fig. 7.5. The relative highest possibility for an accident

operation, ranging from arriving to departure, for example of an oil tanker due to oil spillage.

Further reading

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8

Types of berth structures

8.1 General

The purpose of a berth structure is mainly to provide a vertical front where ships can berth safely. The berth fronts are constructed according to one of the following two main principles, as illustrated in Fig. 8.1.

Solid berth structure: the fill is extended right out to the berth front where a vertical front wall is constructed to resist the horizontal load from the fill and a possible live load on the apron. The solid berth structures can be divided into two main groups, depending on the principle on which the front wall of the structure is constructed in order to obtain sufficient stability:

(a) Gravity wall structure: the front wall of the structure with its own deadweight and bottom friction will be able or self-sufficient to resist the loadings from backfill, useful load and other horizontal and vertical loads acting on the berth wall structure itself.
(b) Sheet pile wall structure: the front wall is not adequate to resist any horizontal loads acting on the structure and must, therefore, be anchored to an anchoring plate, wall or rock behind the berth.

Open berth structure: from the top of a dredged or filled slope and out to the berth front a load-bearing slab is constructed on columns or lamella walls.

A diagrammatic classification of berth structures according to type and construction method is shown in Fig. 8.2.

It is difficult, however, to formulate precise guidelines for the choice of berth type in each individual case. With a view to choosing the

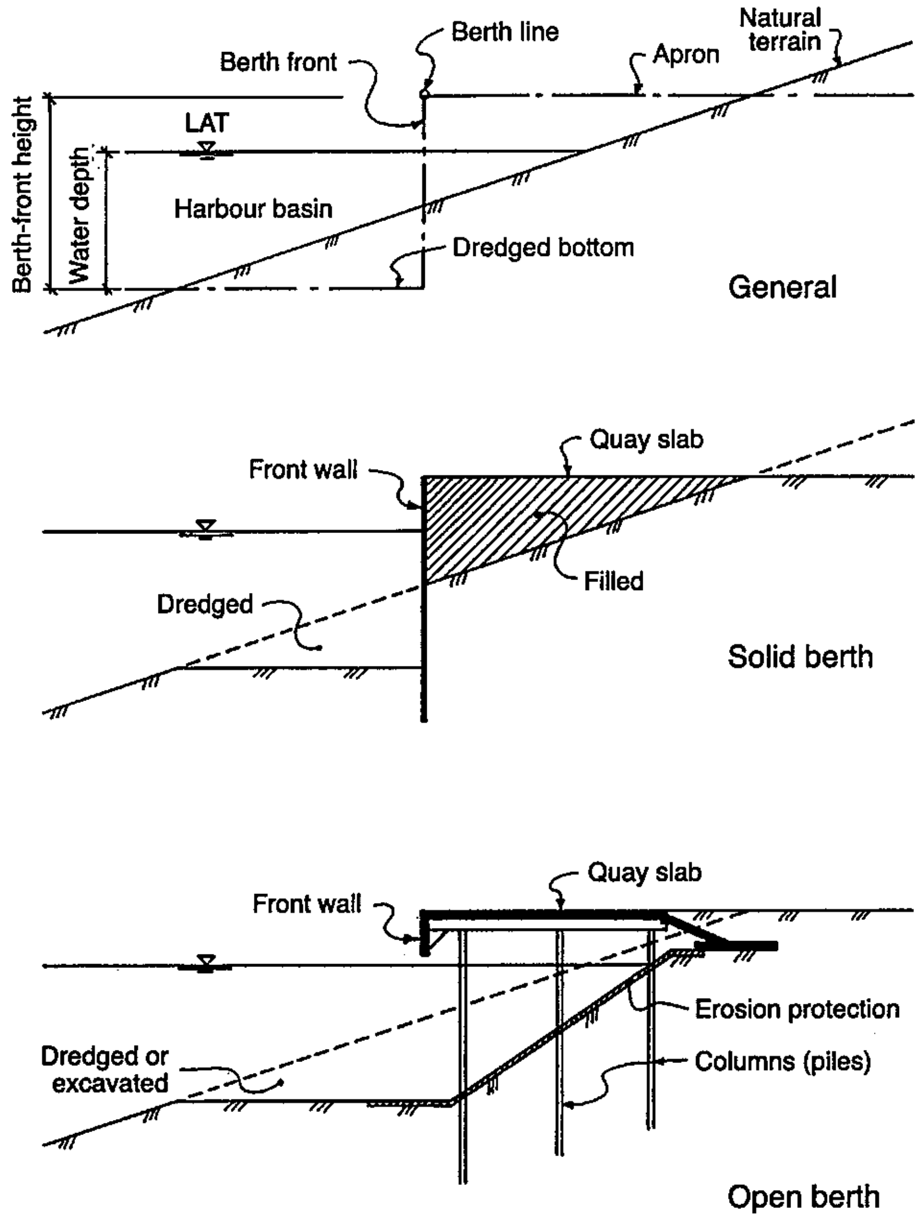

Fig. 8.1. Terminology

technically and economically most favourable type, the factors mentioned in the following paragraphs should be considered.

Berth structures should be designed and constructed to safely resist the vertical loads caused by live loads, trucks, cranes, etc., as well as the horizontal loads from ships' impacts, wind, fill behind the structure, etc.

In general, the solid berth structures are considered more resistant to loadings than the open berth structures, both vertically and horizontally. Since the deadweight of the solid berth structure constitutes a greater part of the total structure weight than the deadweight of open berth structures, the former are less sensitive to overloading. On the

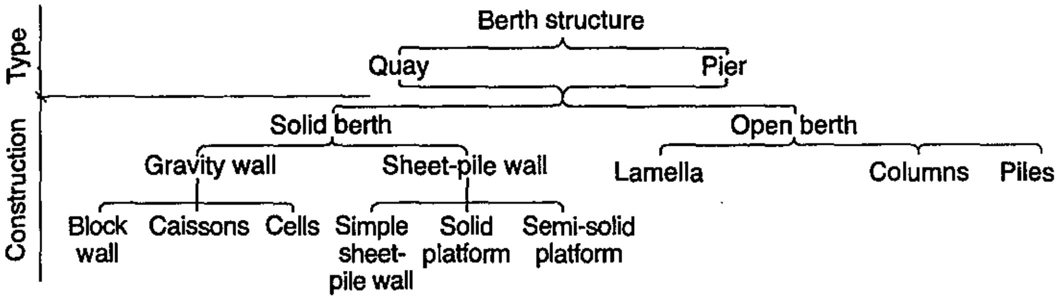

Fig. 8.2. Types of berth structures

other hand, the safety factor applied for solid structures is normally lower than for open structures.

For instance, in an open column berth for ocean-going ships the deadweight of beams and slab is about 15 kN/m^2 berth deck area, while the live load is normally 40 kN/m^2. Such a berth of length 50 m and width 15 m weighs only about 1200 t, but will have to resist the impacts from ships of say 30 000 t displacement or more.

Solid structures are usually more resistant to impact than open structures, i.e. the resistance to impact from ships decreases with increasing slenderness of the structure. For instance, a block wall wharf is far less vulnerable than a pier built as an open berth on piles. An exception to this rule is the open berth on wooden piles where the whole structure is flexible and yields, when ships come alongside, sufficiently to absorb a substantial part of the impact energy.

Top elevation of the berth slab

The top elevation of berth structure should be determined by the following factors:

(a) The elevation of the terminal area behind the berth apron.
(b) The highest observed water level and the tidal level.
(c) The wind-raised water level in the harbour basin.
(d) The wave action in the harbour basin.
(e) The type of ship using the berth.
(f) The harbour installations and the cargo operation.

Generally for cargo berth structure within an impounded dock the top elevation of the berth slab and apron should be at least 1.5 m above the working water level. For berth structure directly connected to the open sea, the top elevation of the berth slab should be 0.5–1.0 m above the highest observed crest of waves in the port depending on the type of cargo handled at the berth.

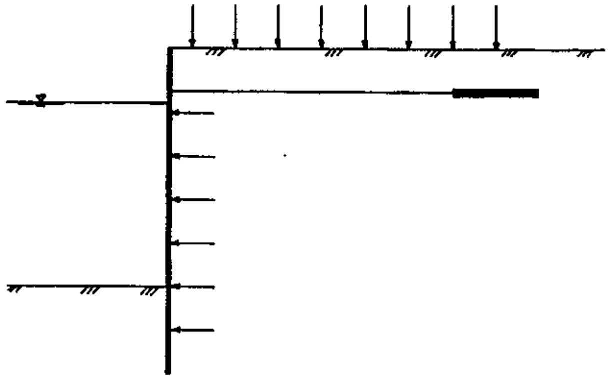

Fig. 8.3. Loading on solid berth

8.2 Vertical loads

As shown in Fig. 8.3, the vertical load on solid structures including live loads, crane loads, etc., will also cause a horizontal load on the front in addition to the load from the fill.

If the height of the berth front and/or the live load are/is large, the influence of the above horizontal load can be reduced or eliminated by building the structure as a solid or semi-solid platform berth, as shown in Fig. 8.4.

In open structures all vertical loads are transmitted by way of the columns or lamella walls to rock, or to a load-resistant sub-soil stratum, as shown in Fig. 8.5.

In a slab/beam structure the vertical loads are taken up by a system similar to a system of beams on elastic supports. The importance of this effect depends on the elasticity and slenderness of the columns, and the properties of the seabed material or the depth to rock. Since the distribution of the loads is determined by the rigidity of the slab and beam system in relation to the spacing of the columns, it

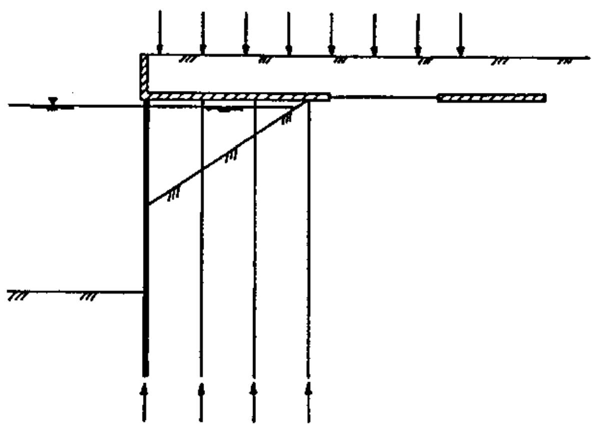

Fig. 8.4. Loading on solid or semi-solid platform berth

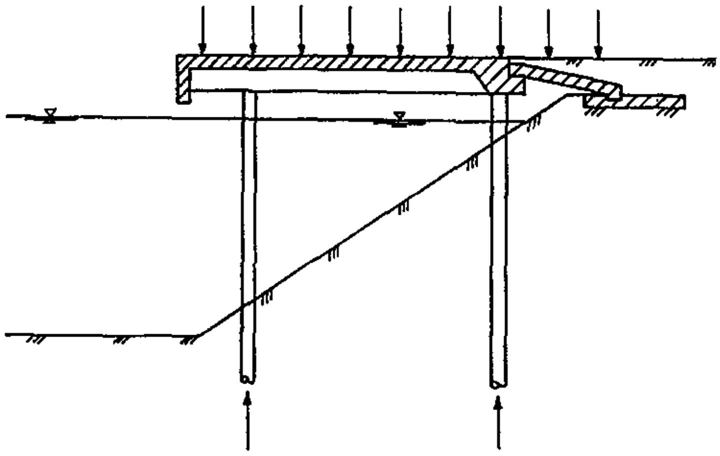

Fig. 8.5. Loading on open berth

can sometimes be recommended that the beams should be made more rigid, enabling a distribution of the loading to take place via a greater number of columns, be investigated.

The columns are considered immovably connected with, and partly fixed to, the beams. The degree of fixation depends on the torsion resistance of the beams. At the bottom of the columns there are various degrees of fixation, depending on the thickness and properties of the seabed material above rock. To assure full fixation is hardly justified in any case. If the seabed layer is only from 0–3 m thick, one must assume that there is a joint at the rock. If the layer is of substantial thickness it is usually correct to assume a joint located 3–5 m below sea bottom, depending on the properties of the seabed material.

8.3 Horizontal loads

The bearing or absorption of the horizontal loads can take place at three levels:

(a) at the berth deck level
(b) between deck and sea bottom levels
(c) at bottom level.

In any case, the bearing of the loads should be arranged as simply and clearly as possible.

8.3.1 At berth deck level

The simplest way of taking up horizontal loads at berth deck level is to brace and anchor the berth deck, as shown in Fig. 8.6, to, if possible, rock behind the berth.

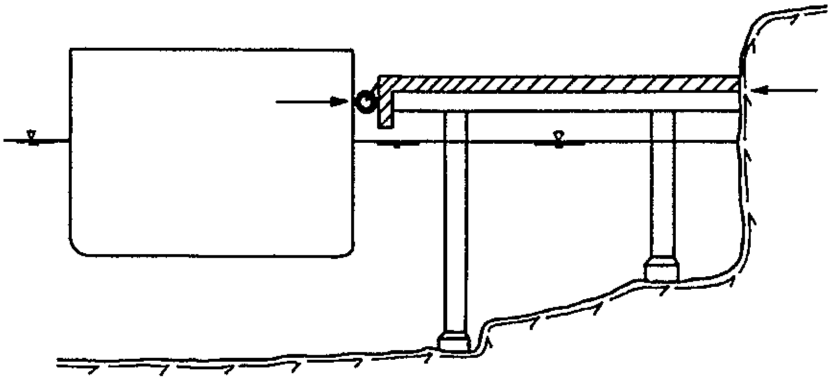

Fig. 8.6. Bracing and anchoring at berth deck level

However, open structures with high and slender columns or piles are sometimes difficult to anchor at berth deck level. Horizontal loads must then be taken up by lamella walls or by anchoring the deck to the ground behind the berth structure.

The horizontal loads from ships' impacts should not be transmitted by way of the columns or piles to the bottom level, assuming rigid frame conditions between deck and columns. The reason is that even if the moments occurring at the column tops could be resisted in theory, there is a great danger of cracks developing in the column tops, i.e. in perhaps the most vulnerable part of the structure. A structural design rule therefore says that columns shall not transfer horizontal loads from ships' impacts.

In an isolated column berth, the horizontal load bearing can be arranged as shown in Fig. 8.7, the berth deck (slab + beams) is here considered as a rigid plate transmitting the ships' impacts to the supports of A and B. From there, all horizontal loads are transmitted by way of the rods 1, 2 and 3 to the immovable shore rock anchors at C, D and E. The anchor bolts in rock must be protected against corrosion, and they must have a length sufficient to provide good anchorage in the rock. A compressive force in the rods would normally not cause any problem, even if the rock were cracked.

The principle behind this type of anchoring is that one of the supports (A) is made immovable in both the longitudinal and the transverse directions of the berth deck, while the other support (B) is designed to allow movements in the longitudinal direction caused by temperature changes and shrinkage.

The berth deck itself will normally be able to resist the horizontal bending moments occurring between A and B due to ships' impacts, although some additional reinforcement along the longer sides of the slab may prove necessary.

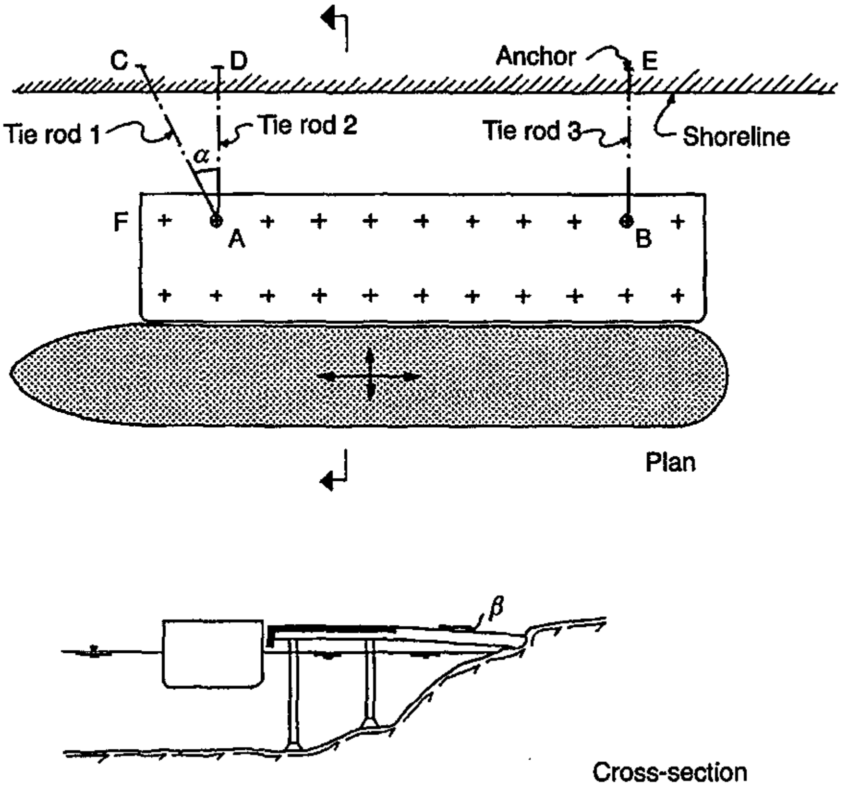

Fig. 8.7. Isolated column berth

The support A can be connected to the anchors C and D by two tie rods, or these rods can be replaced by a trafficable bridge between A–F and C–D, or only rod 2 can be replaced by a bridge, keeping rod 1 as it is. The angle α should be wide, due to the considerable longitudinal forces that can act in the berth deck.

Since the tie rods transmit substantial forces, the connection between rods and berth deck should be arranged in such a way that the rods, the berth beam axis and the column axis meet in one point. This is to avoid secondary moments being set up. As shown in Fig. 8.7 the angle β should be as small as possible so that the vertical component to be transmitted by the column also becomes small.

The tie rods can be very long and they are often supported by separate columns. They must be designed to carry dead load plus some live load and axial loads, and are often shaped as T-beams.

8.3.2 Between slab and sea bottom levels

As mentioned above, the anchors should take up the horizontal forces directly without any forces being transmitted to the columns or piles. Figure 8.8 shows a type of anchorage that is not acceptable because a

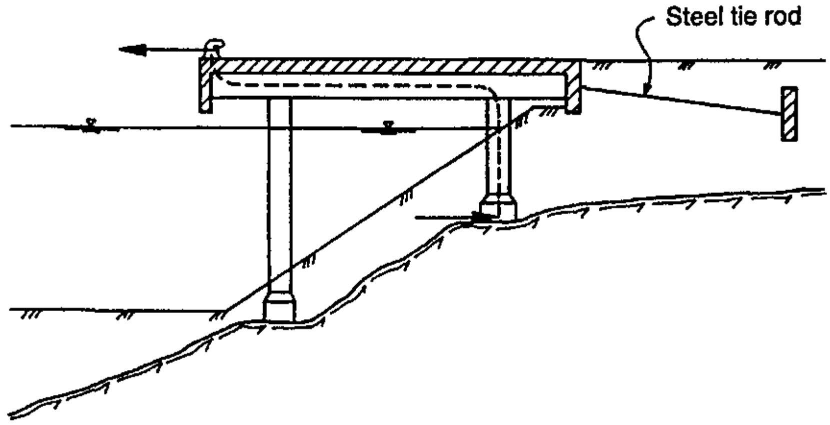

Fig. 8.8. Steel tie rod and retaining wall anchoring

passive soil pressure has to be established in front of the retaining wall before it can absorb the horizontal forces. In other words, the wall must have been subjected to a certain movement first.

In an approximately rigid structure, like the berth on columns shown in Fig. 8.8, the greater part of the horizontal force will be transmitted to the columns and possibly cause some damage to them. The most rigid element of a structure tends to attract the forces acting on the structure. The anchorage should therefore be as shown in Fig. 8.9, consisting of a friction plate and a concrete tie rod, so that the force is transmitted directly to the anchorage.

The anchoring system must be able to take up horizontal forces acting both parallel and perpendicular to the berth front, and must also be designed with a view to the deformations in the longitudinal direction of the berth due to shrinkage and changes of temperature. As shown in Fig. 8.9 the angle β should be as small as possible, and the axes of the tie rod, the beam and the column should intersect in one point.

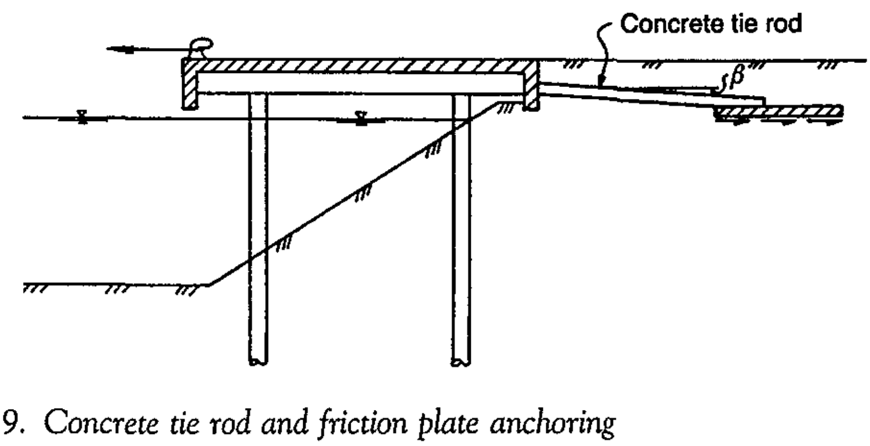

Fig. 8.9. Concrete tie rod and friction plate anchoring

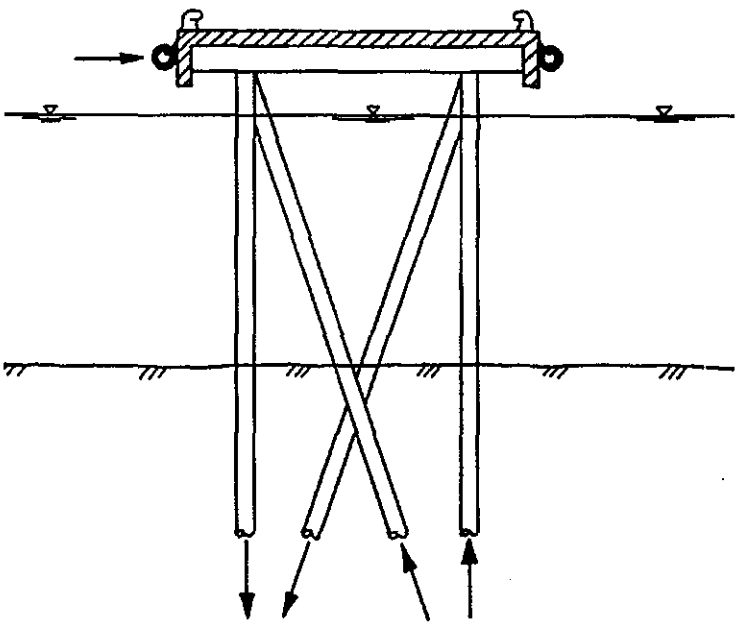

Fig. 8.10. Open pier structure

8.3.3 At bottom level

If the berth structure cannot be anchored at deck level or between deck and sea bottom levels, the forces must be transmitted to the bottom level and be taken up by one of the following means:

(a) batter columns or piles
(b) lamella walls
(c) cells.

One of these means will normally be applied for the bracing of a pier head. Problems are involved in this connection due to the width of the structure being relatively small so that it can be difficult to get a stabilizing moment sufficient to resist the overturning moment. In particular the open structures have a small stabilizing deadweight.

In open structures, where bracing is provided by batter piles, the stabilizing weight of the deck alone is seldom sufficient to take up the horizontal load. Figure 8.10 shows an open pier structure where ships can berth at both sides. The horizontal load is taken up by help of the deadweight of the structure and adherence between piles and soil.

Where sufficient adherence cannot be mobilized, the deadweight can be increased as shown in Fig. 8.11 by adding a layer of sand on top of the slab, or by making the concrete slab thicker.

Where the thickness of the seabed material above rock is too small to provide adequate adherence capacity, a sheet piling cell can provide an alternative for taking up horizontal loads. At the head of

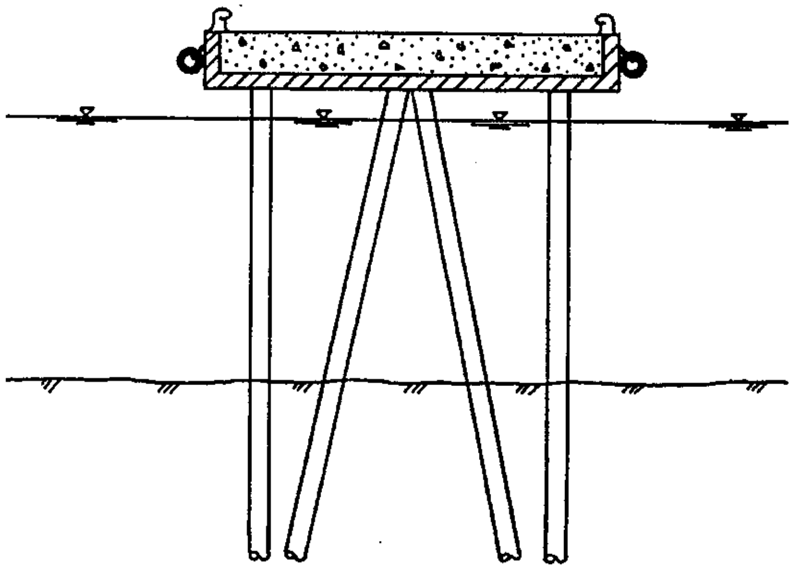

Fig. 8.11. Increased deadweight

an important berth, construction of two cells should be considered for safety reasons.

If the sea bottom is rock, a lamella wall at the end of the open structure can be a good alternative, as shown in Fig. 8.12. The stabilizing moment from the deadweight of wall and deck should be greater than the overturning moment from the horizontal load. Rock anchor bolts or post-tensioned anchor cables bored into the rock should give the necessary additional safety against overturning. The bolts or rods should be designed to allow for possible corrosion. Such bolts should have a factor of safety of at least 2 based on their net cross-sectional dimensions after corrosion, and the amount of rock that will be

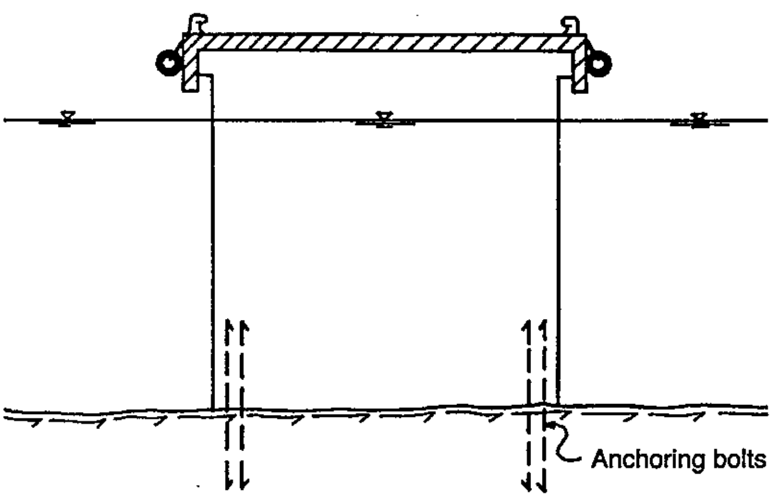

Fig. 8.12. Anchoring through lamella wall

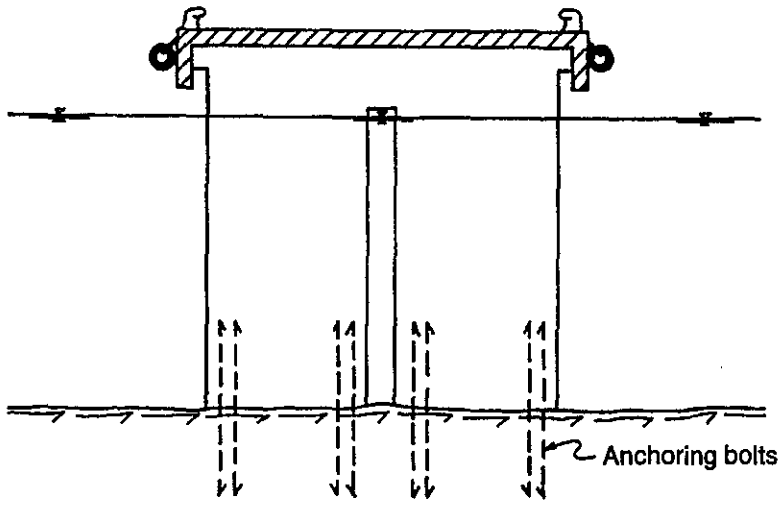

Fig. 8.13. Frame wall

involved. In order to simplify the pouring of concrete underwater, the wall can be designed as a frame wall, as shown in Fig. 8.13.

Where the horizontal load is taken up by one of the above methods, it is economically best to involve as few load transfer points as possible and design the columns for axial loads only, as shown in Fig. 8.14.

If extremely great impacts possibly may occur once or twice during the lifetime of a structure, this should be taken into consideration. For instance, a dolphin could be designed to take up normal loads, while its foundation could be designed in such a way that it permits the dolphin to skid along the foundation under an extreme load. It would probably be the cheapest solution to construct the dolphin in

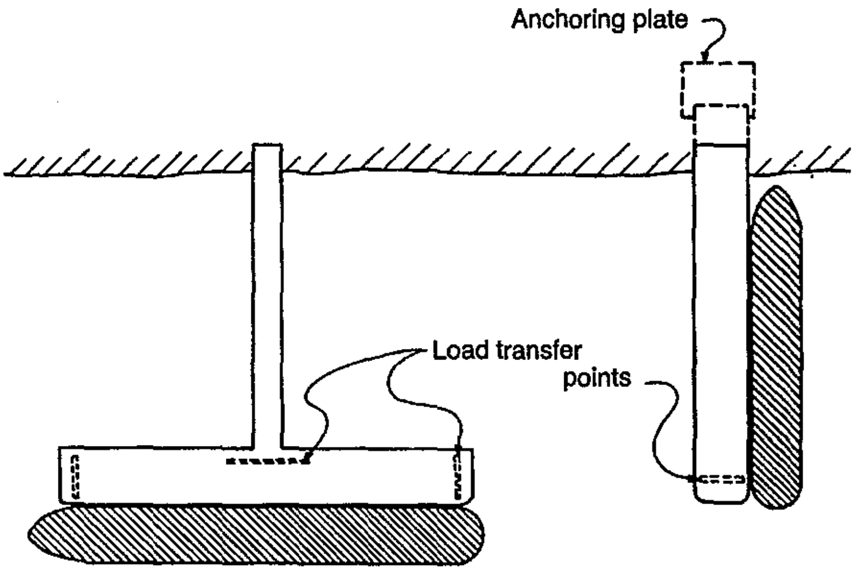

Fig. 8.14. There should be as few load transfer points as possible

this way and, in that case, jack it back into its original position after a possible skidding. To construct the whole structure to resist possible extreme loads once or twice in the berth structure's lifetime would in many cases involve prohibitive expenses.

8.4 Factors affecting the choice of structures

8.4.1 Soil conditions

The fact that soil conditions can vary very much from one site to another has led to the development of a wide spectrum of types of berth structures. If the soil is loose and has a low bearing capacity it would serve no useful purpose to think of constructing a solid block wall type of structure. In such a case it would be better to consider an open type founded on piles driven down to rock or to another sufficiently firm stratum.

In other words, reliable and complete soil investigations must be carried out at the site of the new berth structure. The soil engineer should normally be consulted about the type of foundation to be chosen.

8.4.2 Underwater work

Avoidance of construction work that must be carried out underwater is an important goal in modern berth design. Emphasis is placed on the application of construction methods that allow as much work as possible to be carried out from a position above the water, thus keeping the amount of diving at a minimum. Sheet pile structures and structures founded on steel pipe piles are ideal in this respect.

One reason why diving work should be eliminated is that one or two divers only carry out important structural elements, where a close supervision of their work is difficult to accomplish. The working possibilities underwater are often limited as compared to the situation above water, and so is the visibility. The work that has to be carried out by divers, if any, should therefore be of a simple nature. When underwater work is unavoidable the structural engineer should consult an experienced diver in advance on the working methods to be applied.

8.4.3 Wave action

Open berth structures are normally more favourable than solid ones in respect of the reflection of incoming waves against the berth front. At an open structure the waves will be damped to a great extent against the rough rubble-covered slope. At the vertical wall of a solid berth any wave reflections and other disturbances can have harmful effects, particularly if the shape of the harbour basin is such that it supports wave reflection.

8.4.4 Design experience

Particular qualifications are required from those who are entrusted with responsibility for the design and construction of marine structures. Most of the construction work takes place in connection with water and therefore working techniques, plant and machinery are, to a considerable extent, different from those applied in construction on land.

Documented relevant experience among the engineers and other personnel appointed for the maritime sector is therefore a requirement. In the Norwegian Concrete Association's Guidelines for Design and Construction of Concrete Structures in Marine Environments (Norwegian Concrete Association, 2003) this requirement is emphasized.

8.4.5 Construction equipment

When designing a berth structure, thought should be given to which types of construction plant and machinery can reasonably be procured for the site in question. One should also keep in mind that a number of contractors ought to be able to procure the necessary equipment so that true competition is secured.

It is from time to time argued that various contractors own different types of equipment and therefore tend to practise construction methods deviating from contractual specifications. However, a closer study of the methods used by the most experienced contractors shows that their methods are very much the same. This fact should be considered by the structural engineers and could possibly lead to a certain degree of standardization in this sector.

The heavy equipment used in foundation works does not, on the other hand, easily lend itself to standardization. This equipment usually involves high transportation and installation costs. Therefore, alternative foundation methods are sometimes considered with a view to the utilization of equipment that is found locally. The magnitude of the foundation works on each particular project is also a factor to consider in this respect.

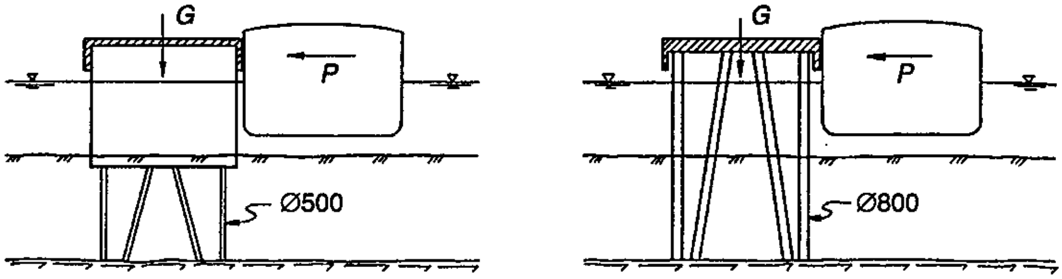

Fig. 8.15. Cross-section of a pier

8.4.6 Materials

A berth structure can be constructed out of timber, steel and concrete or a combination of these materials. The general choice of construction materials to be adopted will be dependent on the purpose of the structure and economic considerations. The durability under marine environmental conditions is of particular importance for marine structures. The aggressive action from seawater requires special attention.

The possibility of actually getting the specified materials delivered to the site is a matter which must be carefully investigated. A modification of the structural system can be the result of such investigation.

Figure 8.15 shows the cross-section of a pier where the horizontal load *P* is resisted by stabilizing lamella walls founded on Ø 500 mm steel piles driven to rock. It could have been cheaper to use Ø 800 mm piles extended up to the berth slab, and to increase the slab thickness to obtain sufficient weight G. However, in this case one was not able to get Ø 800 mm pipes from any source in time, and the Ø 500 mm pipes and lamella walls had to be used instead.

8.4.7 Construction time

If a new ferry berth is to be built on the same place as an old one that is still in operation, or a new berth structure is going to be erected close to an existing berth, the operation of which would be hampered during the construction period, a timesaving construction method should be emphasized even if its construction cost is higher.

Among the solid berth structures, the types called 'cells', 'simple steel pile wall' and 'solid platform' require a construction time of 3–5 lin m of berth front per week. For a lamella berth structure the performance is about 15 m^2 slab per week, and for column berths 30–60 m^2 structure slab per week. In quite special circumstances, and/or when using an advanced formwork system, a production of 150 m^2 of slab per week

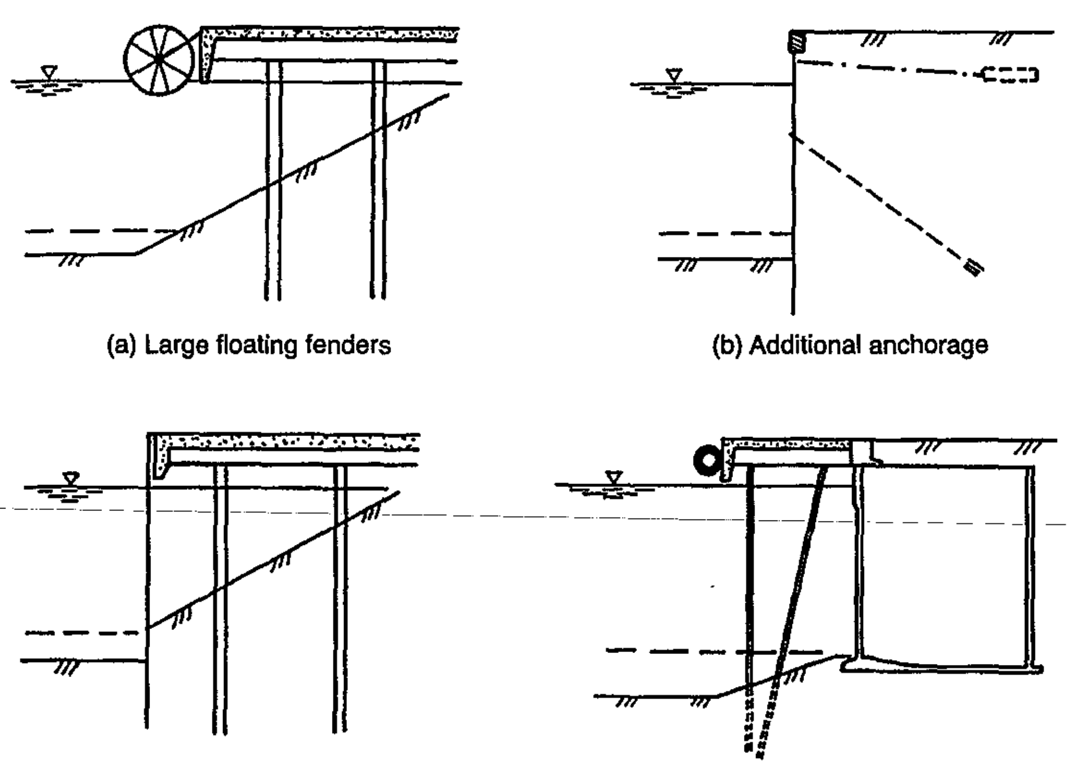

Fig. 8.16. Principle of increasing the water depth

may be achieved in a column or pile structure project. These figures are based on one working team working 8 h/day.

8.4.8 Future extensions

Usually, provisions should be made for possible future extensions of the berth in one or more directions. There can be a need for increase of the water depth in front of existing berth structures to provide greater depths along the front due to the increase in the size of ships over the last years. This can create and has created problems for the stability of the front of the berth structure. The following methods, shown in Fig. 8.16, show, in principle, some solutions to obtain greater depths in the front while utilizing as much of the existing berth structure as possible:

(a) Large floating fenders between the ship and the berth structure. The solution is cheap from a construction point of view but there can be problems with the reach of cranes, etc.
(b) Additional anchorage of the existing sheet pile structure with new grout anchors or a reduction of the live load on the quay loading area, or replacing the existing filling behind the quay front by lighter materials.

(c) Sheet piling in front of the existing berth structure.
(d) A new berth structure in front of the existing berth structure. This type of solution can offer more possibilities than cases (a), (b) and (c) because this solution gives additional area to the apron and could be designed for higher live loads.

8.4.9 Expansion joints

Depending upon the type of harbour structure, the soil condition and temperature variations, expansion joints must be provided in order to accommodate the movements arising from temperature changes, shrinkage and some yielding of the foundations. Steel and/or reinforced concrete structures should be designed to prevent temperature and shrinkage cracks.

The expansion joints in the sections should be keyed for mutual transfer of shear forces and should be so designed that changes in the length of the sections are not hindered. The expansion joints should, especially for solid berth structures, be covered to prevent the backfill from being washed out.

If the berth slab exceeds a certain length, expansion joints must be provided at certain intervals perpendicular to the quay front. Usually their spacing is about 60 m, but this depends very much on the system used for the lengthwise anchoring of the berth. If a lamella wall at the middle of the berth stretch anchors the berth, it is quite possible to make the berth 100–200 m long without providing any expansion joint. Horizontal forces acting perpendicular to the joints have to be absorbed by some kind of indentation.

8.4.10 Construction costs

Construction unit prices tend to vary from one part of the country to the other, and will also depend on the competition existing among the contractors at any time.

Generally it can be said that open berth structures are relatively cheaper than solid structures the greater is the water depth at the front.

8.5 Norwegian and international berth construction

In several ways Norwegian berth construction methods differ from international practice. This is partly because in Norway the authorities permit the construction of slender load bearing concrete column structures poured underwater. An experienced contractor is able to construct, in this way, columns of length up to about 30 m and diameter about 90–100 cm. Tests carried out on such structures have shown that they are fully intact 60 years after pouring.

This method of design and construction of berths founded on slender concrete elements poured under water represents a pioneer work developed in Norway by engineers in the last three generations. In the international market one will see much heavier berth structures built for similar purposes under similar conditions. Foreign port contractors who have designed berths in Norway have often been faced with the Norwegian contractors alternative designs of the above type, which are faster and cheaper to build.

In Norway, on average during the last 10 years, approximately 20 per cent of all berth structures have been built as solid berth structures and approximately 80 per cent have been built as open berth structures. The reason for these figures is mainly due to geotechnical and geological conditions. Berth structures founded on steel pipe piles that are reinforced and filled with concrete after driving represent a type that has been successfully developed in Norway.

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9

Gravity-wall structures

9.1 General

This type of structure can be sub-divided into the following three groups depending on type of structural design:

(a) block wall berths
(b) caisson berths
(c) cell berths.

The gravity berth wall structure may generally only be used where the seabed is good and the risk for settlement is low.

9.2 Block wall berths

Block wall berths belong to the oldest type of berth structures. They consist of large blocks placed one upon the other in a masonry wall pattern, as shown in Fig. 9.1. Such berths, built on firm ground with blocks of good-quality natural stone or concrete, are structures of long life, and require only modest maintenance. Due to the present high costs of mining natural stone blocks, only concrete blocks can be considered economical for projects nowadays.

Since a great deal of the construction work has to be carried out underwater by divers, the construction costs are usually very high. Only special local conditions could therefore justify the use of this type of structure nowadays. Such conditions could involve, for instance, a long berth to be founded on very firm ground, and, where cheap unskilled labour could be engaged, in casting a sufficient number of concrete blocks before the start of the actual construction. Thus any

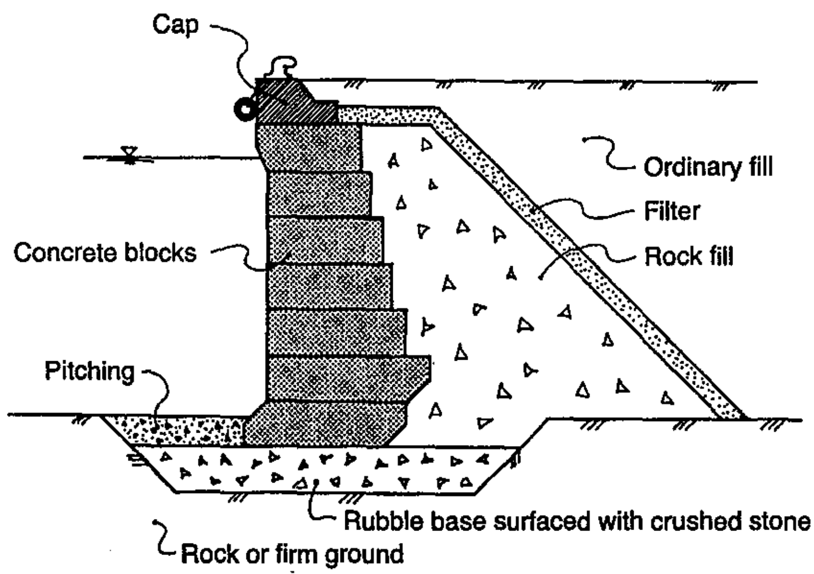


Fig. 9.1. Block wall berth

idle time for the skilled labour will be minimized. In order to minimize the extent of underwater work, the blocks should be of equal size, as far as possible, and, after casting the blocks, each course should first be arranged and marked onshore in order to facilitate its final placing in the water.

To ensure the stability of the individual blocks they ought to be so big that the maximum capacity of the block-handling equipment (cranes, etc.) is fully utilized. The size of concrete blocks is also determined by the available casting equipment and storage space for the blocks. Their weight may vary from about 150–2000 kN. Blocks of natural stone normally had weights from 150–500 kN, depending on the distance to the quarry and the transport equipment.

Due to their great deadweight, block wall berths should be used only on very firm ground in order to avoid settlement. The structure can cause great stresses at the outer edge of the bottom course of the wall, which should therefore be laid on a rubble-base surface levelled with crushed stone. The ideal foundation is achieved where the wall can be laid directly on rock levelled with an in situ concrete footing.

Figure 9.2 shows a method of improving the ground condition by dredging away the clay layer above the rock and then replacing the dredged clay with a filling of sand or gravel. To reduce the settlement in the sand, a vibro-compaction method is used. Vibro-compaction is an in situ method of compaction of loose cohesionless granular soils, like sand and gravel. The process is based on the principle that granular soils below maximum density are compacted under the influence of vibration motion.

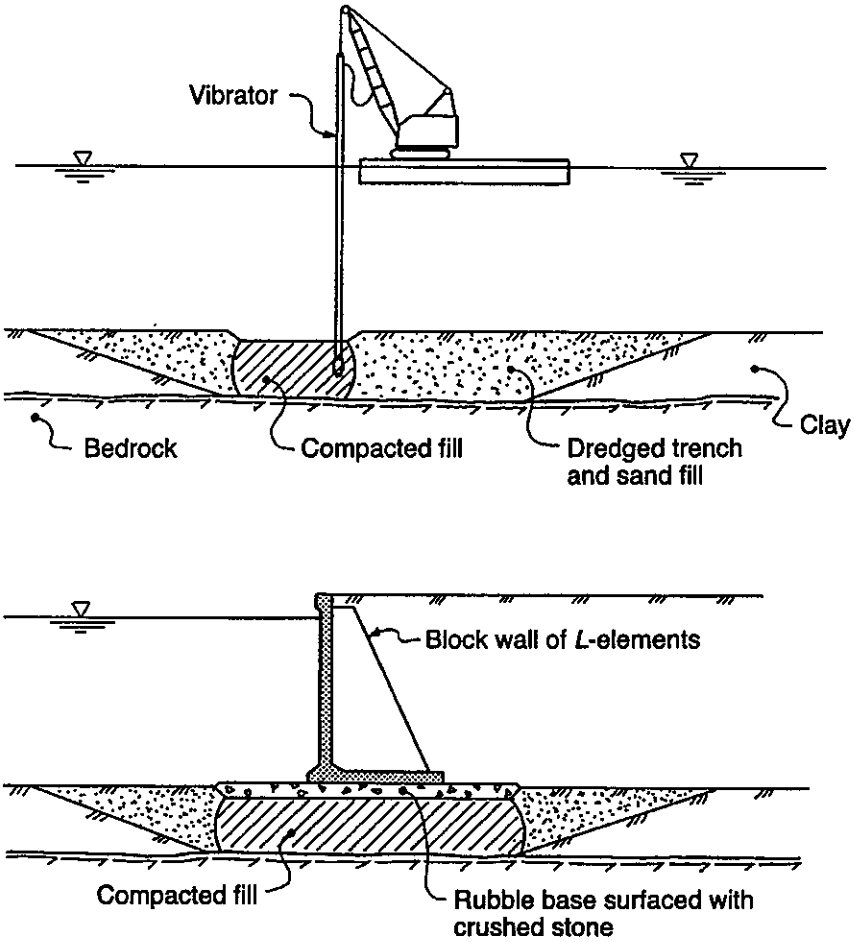


Fig. 9.2. Improving the ground condition by vibro-compaction

Another method of achieving improvement and reinforcement of soft cohesive soils is by the installation of stone columns to carry structural loads as shown in Fig. 9.3. The stone columns will improve the bearing capacity of the foundation and will reduce the overall settlement, as they are stiffer than the soil they have replaced. This method is called vibro-replacement, and can generally use the same basic vibratory equipment as the vibro-compaction method.

A modified method of constructing block wall berths has been developed. Instead of using concrete blocks, reinforced concrete retaining wall elements have been developed. These L-elements or L-blocks have been constructed in the same way as concrete caissons on the shore, but transferred and installed at the berth site by cranes. The berth wall is made by installing the L-elements side by side in position on a prepared gravel and/or rubble base at the sea bottom. The elements are shown in Fig. 9.4. Elements without ribs are constructed with a maximum height of about 7 m, and with ribs the height of the elements has been up to about 20 m. The length of the

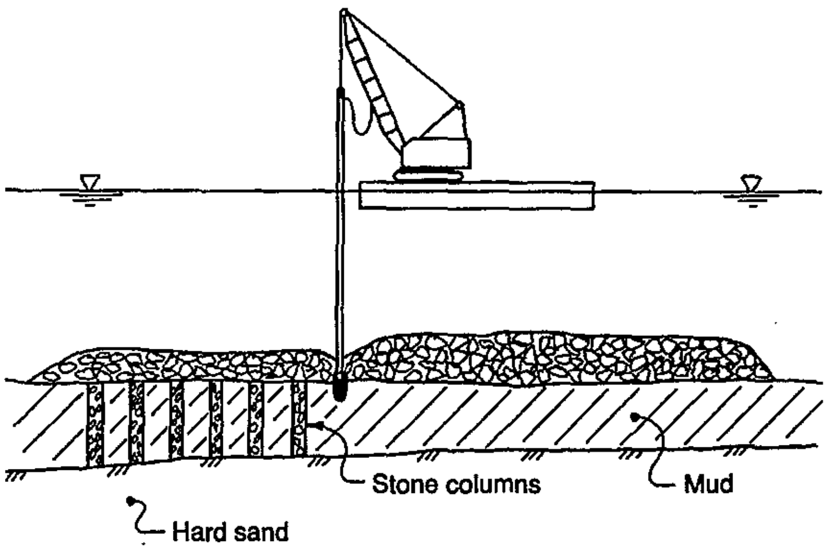


Fig. 9.3. Improving the ground condition by vibro-replacement

elements has varied between 3–12 m, depending on the capacity of the mobile or floating cranes. Mobile cranes can have a lifting capacity of up to 100 tons, but due to the availability of equipment, a more practical limit is about 30 tons. Floating port cranes can have lifting capacity of about 200 tons, but special heavy floating cranes exist with a lifting capacity of about 800 tons.

An article by Lauri Pitkälä, Finland, printed in PIANC Bulletin No. 54 (PIANC, 1986), describes in detail the construction of berth structures with L-elements. The typical building order of a berth structure with L-elements is shown in Fig. 9.5. The usual tolerances for element installation are a deviation in the x, y and z directions of 50 mm, inclinations of 1:400 and angular misalignment of 0.5°.

The fill placed at the back of the wall ought to have as great a frictional angle as possible in order to reduce the lateral earth pressure. Nearest to the wall the fill should consist of stone or crushed rock, while

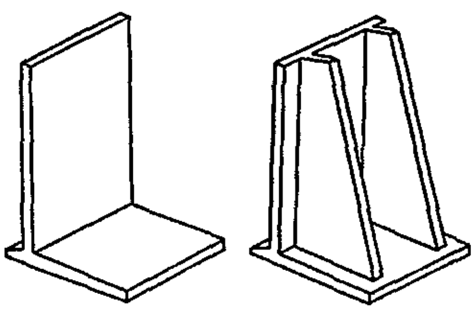


Fig. 9.4. L-element

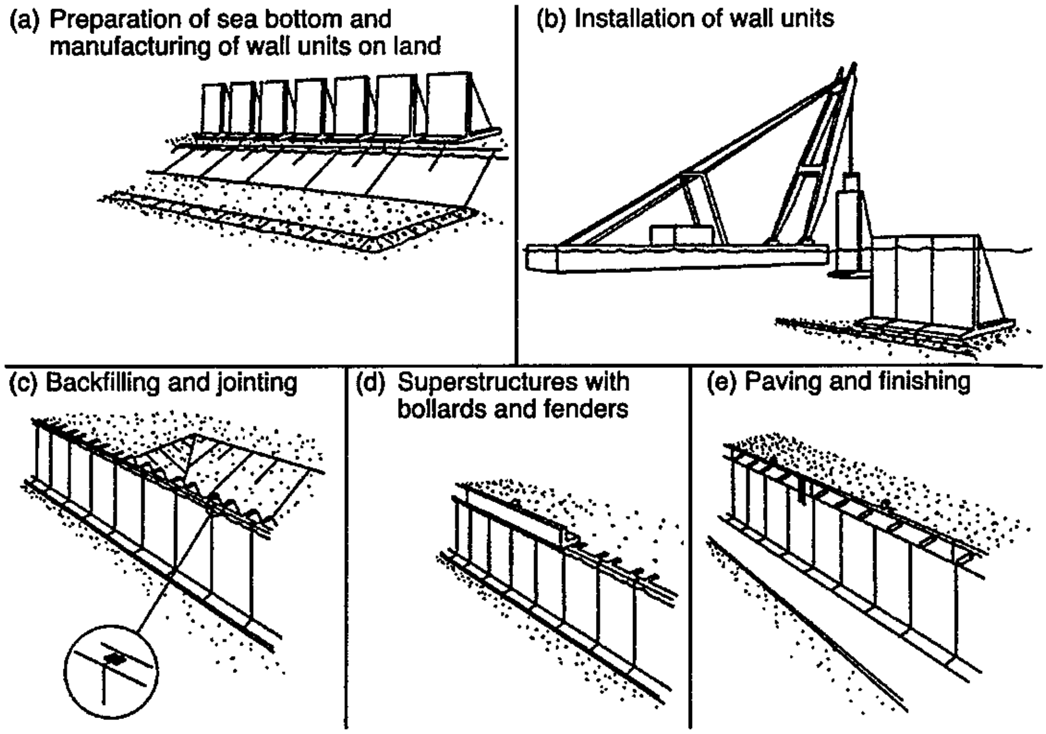


Fig. 9.5. Construction of L-element wall

finer or mixed fill can be used further back. Between the coarser and the finer masses there must be a filter preventing the penetration of the finer material into the rock fill. Above the low-water level all the fill should be compacted.

When the placing of the blocks or elements has been completed, the wall should be left for a while to settle before an in situ reinforced concrete cap (capping beam) is placed on top of the wall. This cap will keep the blocks of the top course in place and also provide a base for the installation of bollards, a quay-front kerb and other equipment.

9.3 Caisson berths

In caisson quays the berth front is established by the placing of precast concrete caissons in a row corresponding to the planned alignment of the new berth. The caissons may be differently shaped and designed, depending on the site conditions and the available construction equipment. Rectangular caissons are the most usual. See Fig. 9.6.

The caissons are usually made ashore and then launched, towed out and sunk in position on a prepared gravel and/or rubble base. Thus, the underwater work is reduced to a minimum. It is both very economic and convenient if the caissons can be made on an existing slipway or in a dry

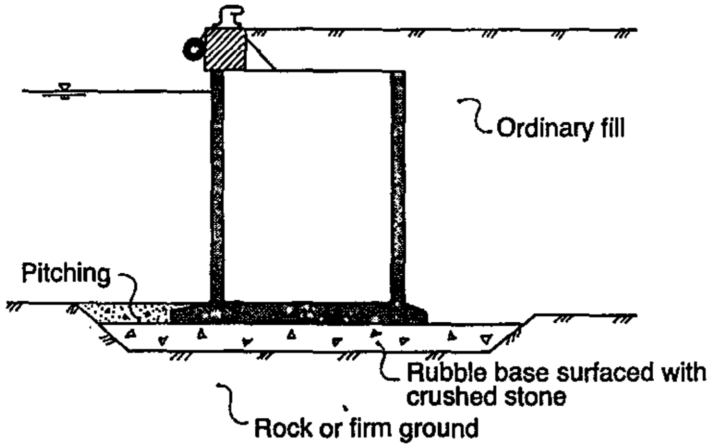


Fig. 9.6. Caisson berth

dock, from which they can easily be launched. For economic reasons, the caissons should also preferably be made in a considerable number so that the production can be arranged in a rational way with multi-employment of the formwork units.

For convenience of construction, launching, towing, placing, etc. of the caissons, experience has shown that the caisson dimensions for economic reasons usually should not be larger than about 30 m long, 25 m wide and 20 m high. The caissons should be designed for all stages during construction and service.

The caissons are usually placed on a firm base of gravel and/or rubble, well compacted and accurately levelled. It is very important that before placing of the caissons, most of the settlements are brought to a minimum, particularly any uneven settlement. If the site is exposed to waves and currents, the base and the caissons should be designed in such a way that the time required for launching, towing and placing of the caissons is as short as possible. After the placing of the caissons they are filled with suitable material, and a reinforced concrete cap is provided for the top, as is done on block wall berths.

In caisson berths it is easier to reduce the stresses at the outer edge of the caisson foot than is the case for block wall berths. Increasing the width of the caisson or providing it with two or three chambers of which only the rear chambers are filled, as shown in Fig. 9.7, can reduce the stresses. The caissons must be designed to also resist the loads and stresses occurring during production, launching, towing, placing and filling.

All joints between the caissons must be sealed if the caissons are used to retain materials behind them and/or prevent waves or current from passing through the gaps between them. The joints should be designed

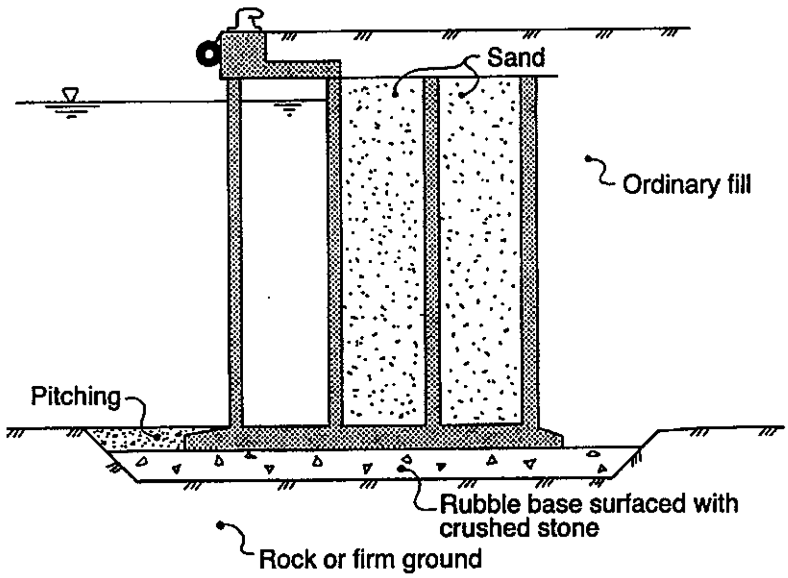


Fig. 9.7. Caisson berth with three chambers

for placing tolerances and uneven settlements. The placing tolerances should be ±150 mm in sheltered water.

9.4 Cell berths

During recent years sheet pile cell berths have become one of the most used types of gravity wall berth. One of the main reasons for this is the increasing ratio of the cost of labour (divers, etc.) to the cost of material, as compared to the construction of block wall and caisson berths. Various geometric configurations of sheet pile cell berths are used, but the usual designs are shown in Fig. 9.8.

Circular main cells connected with arched cells are the most used form of construction. The circular cells have the advantage that each cell can be individually constructed and filled and are, therefore, independently

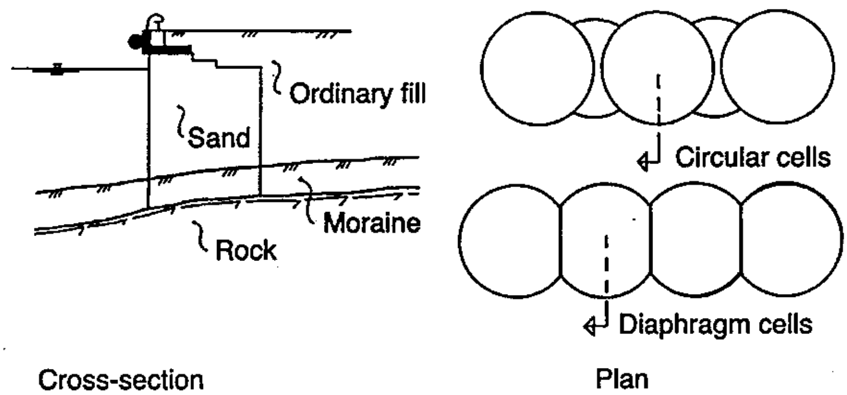
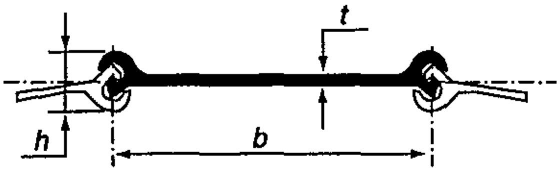


Fig. 9.8. Sheet pile berth



Section	b (mm)	h (mm)	t (mm)	s (mm)	Coating area* m²/m² wall	Steel section (cm²/m)	Mass kg/m single pile	Mass kg/m² wall
AS 500–12.0	500*	92	12.0		1.15**	94.6**	74.3	149
AS 500–12.5	500*	92	12.5		1.15**	97.2**	76.3	153
AS 500–12.7	500*	92	12.7		1.15**	98.2**	77.1	154

Section	Section modulus	Moment of inertia	Radius of gyration	Bending moment capacity at yield point		
	cm²/m wall	cm²/m wall	cm/m wall	S 240 GP	S 355 GP (kN m/m)	S 430 GP
AS 500–12.0	51**	196**		Interlock strength up to 6000 kN/m		
AS 500–12.5	51**	201**				
AS 500–12.7	52**	204**				

Notes
* The effective width to be taken into account for design purposes (lay-out) is 503 mm for all AS 500 sheet piles.
** Per single pile.

Fig. 9.9. Straight web sections from Arcelor, Luxembourg

stable. In the diaphragm cells, or flat cells' design, shown in the figure, sheet pile archs are connected with straight sheet pile walls.

Circular steel sheet pile cells with large diameters are one of the most used berth structures in the arctic, with ice-affected waters, since this type of berth structure can resist large horizontal forces.

A cell berth consists of a row of cells filled with sand, gravel, etc., connected as shown in Fig. 9.8. The diameter of the main cells normally varies from 10–20 m. The circular and arched cells are formed by flat steel sheet piles of width 40 or 50 cm, web thickness 9–12.7 mm and weight per m² 128–154 kp. Figure 9.9 shows straight web sections from Arcelor, Luxembourg. The sheet pile cells offer the advantage that they can be designed as stable gravity walls without wale and anchoring. The nature of the cell fill material must be carefully specified and controlled. Experience has shown that the permeability of the sheet pile cells interlocking under tension is low, so the need for drainage through the cell constructions should be carefully investigated.

The sheet piles are joined by lock arrangements acting in tension to retain the fill inside the cell. The locks have a guaranteed interlock tensile strength from 2000 to 6000 kN/lin m depending on the type of steel and sections. The interlocks between the flat steel sheet piles may have either one-point or three-point contacts depending on the manufacturer of the sheet pile. The sheet pile profiles can resist an angular distortion of about 10° in the locks. Tables indicating cell diameters, distances to the neighbouring cells, dimensions of intermediate arch, etc. are published by the manufacturers of these profiles. An example of circular cells from Hoech Stahl AG, Germany is given in Fig. 9.10.

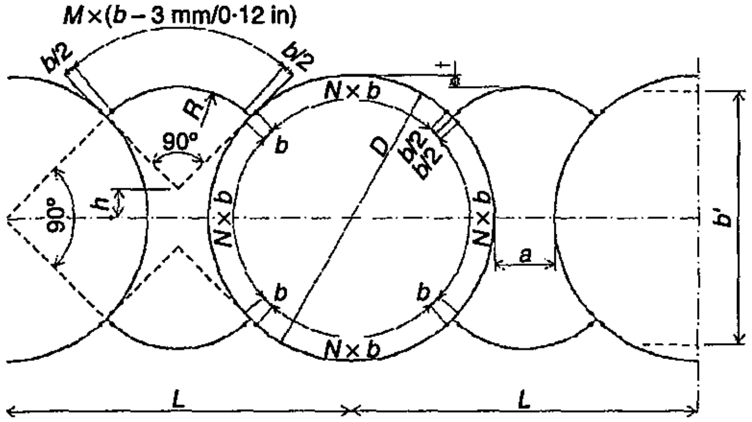
The choice of cell diameter depends on the water depth and the loads. The sheet pile area or the sheet pile weight/lin m berth front is virtually independent of the cell diameter. The unit price of the fill in the cells is therefore often decisive for the construction cost.

The depth to which the sheet piles are to be driven may be reduced according to the stair–step method in order to save steel, as shown in Fig. 9.11. This method can be used provided that the backfill material is of high quality and the angle α is less than 15°.

The design of cell structures is based on both the conventional theory of structures and empirical formulas, and in the literature one will find slightly different calculation methods described. As illustrated in Fig. 9.12, the cell structure must be designed to resist the following modes of failure:

(a) Tilting due to external loading of the cell structure. Normally this will give the minimum cell diameter.
(b) Tilting due to failure in vertical or horizontal shear within the fill material.
(c) Tension failure in the sheet pile locks. Normally this will give the maximum cell diameter.
(d) Horizontal sliding of the cell structure.
(e) Tilting due to rotational failure on a curved rupture surface at or near the base of the cell structure.
(f) General shear failure of the soil beneath the cell structure.
(g) Settlement of the cell structure.

If the sheet pile cell structures have been driven into soil below the dredged or sea-bottom level, the resistance due to tilting, sliding, rotational failure etc. of the structure will be provided by the passive resistance of the soil below the sea bottom and the pullout resistance of the sheet piles on the landward side.



② C = number of piles on the circumference C = 4 (N + 1)

③ Uneven number

④ Mean width of an equal rectangular area

Circular cell walls				Diaphragm walls					Analytical width	Diam. of guide table
C ②	N ③	D	L	M	R	h	t	a	b ④	d
no.	no.	ft m	ft m	no.	ft m	ft m	ft m	ft m	ft m	ft m
				Sections FL 511, FL 512, FL 512.7						
56	13	29.24 8.91	35.45 10.80	9	10.44 3.18	2.95 0.90	1.22 0.37	6.20 1.89	25.38 7.89	28.67 8.74
64	15	33.42 10.19	36.40 11.70	9	10.44 3.18	4.43 1.35	1.84 0.56	4.98 1.52	29.40 8.96	32.82 10.00
72	17	37.60 11.46	41.35 12.60	9	10.44 3.18	5.91 1.80	2.45 0.75	3.76 1.15	32.97 10.05	36.98 11.27
80	19	41.77 12.73	47.26 14.41	11	12.53 3.82	5.91 1.80	2.45 0.75	5.49 1.67	36.70 11.19	41.13 12.54
88	21	45.95 14.01	50.21 15.31	11	12.53 3.82	7.38 2.25	3.06 0.93	4.26 1.30	40.28 12.28	45.28 13.80
96	23	50.13 15.28	56.12 17.11	13	14.62 4.46	7.38 2.25	3.06 0.93	5.99 1.83	44.01 13.41	49.43 15.07
104	25	54.31 16.55	59.08 18.81	13	14.62 4.46	8.86 2.70	3.67 1.12	4.77 1.45	47.59 14.50	53.59 16.33
112	27	58.48 17.83	64.98 19.81	15	16.71 5.09	8.86 2.70	3.67 1.12	6.50 1.98	51.32 15.64	57.74 17.60
120	29	62.66 19.10	67.94 20.71	15	16.71 5.09	10.34 3.15	4.28 1.31	5.28 1.61	54.90 16.73	61.89 18.86
128	31	66.84 20.37	70.89 21.61	15	16.71 5.09	11.82 3.60	4.89 1.49	4.05 1.24	58.50 17.83	66.04 20.13
136	33	71.01 21.65	76.80 23.41	17	18.80 5.73	11.82 3.60	4.89 1.49	5.78 1.76	62.21 18.96	70.20 21.40

Fig. 9.10. Flat steel sheet pile profiles from Hoech Stahl AG

The cylindrical cell type of berth is the easiest to build since each cell is stable by itself when filled with sand, gravel, etc. The finished cell can then be utilized as a working platform for the further work. The arched cells between the cylinders must be filled after the filling of the

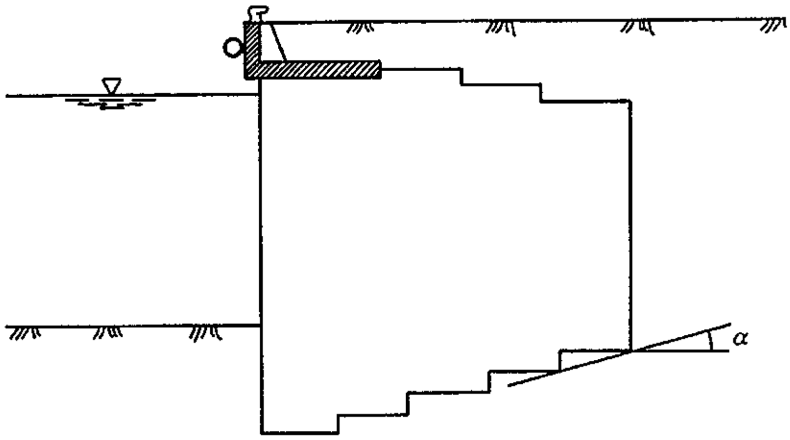


Fig. 9.11. Stair–step method for sheet piling

cylinders. The cellular structure is able to resist horizontal forces without the assistance of any anchoring. The cells can therefore be built independently of the backfill. They are also suitable as supporting structures such as dolphins and head sections in piers.

For the construction of a cell berth consisting of, for instance, three circular main cells and two intermediate arched cells, as shown in Fig. 9.8, the procedure will be as follows:

(a) The template for the first main cell is erected in position.
(b) The sheet piles for connecting the main cell with the arched cells are put in position.
(c) The sheet piles between the above connecting piles for the arched cells are put in position and the main cell is closed. In general, the whole cell must be set and closed before driving of the piles, and it is presupposed that the initial slight penetration of the sheet piles into the soil is gained by their own deadweight.
(d) The first main cell is driven to firm ground or rock. The driving itself should be carried out in several stages proceeding around the cell circumference and always limiting the penetration to between 1.0–2.0 m.
(e) The first main cell template is removed.
(f) The first main cell is filled immediately.
(g) Points (a) through (f) are carried out for the second main cell while the first cell is being filled according to point (f).
(h) Points (a) through (f) are carried out for the third main cell during the filling of the second cell.
(i) The template for the first arched cell (between the first and second main cells) is placed when the filling of the second main cell is finished.
(j) The sheet piles for the first arched cell are put in position and driven.

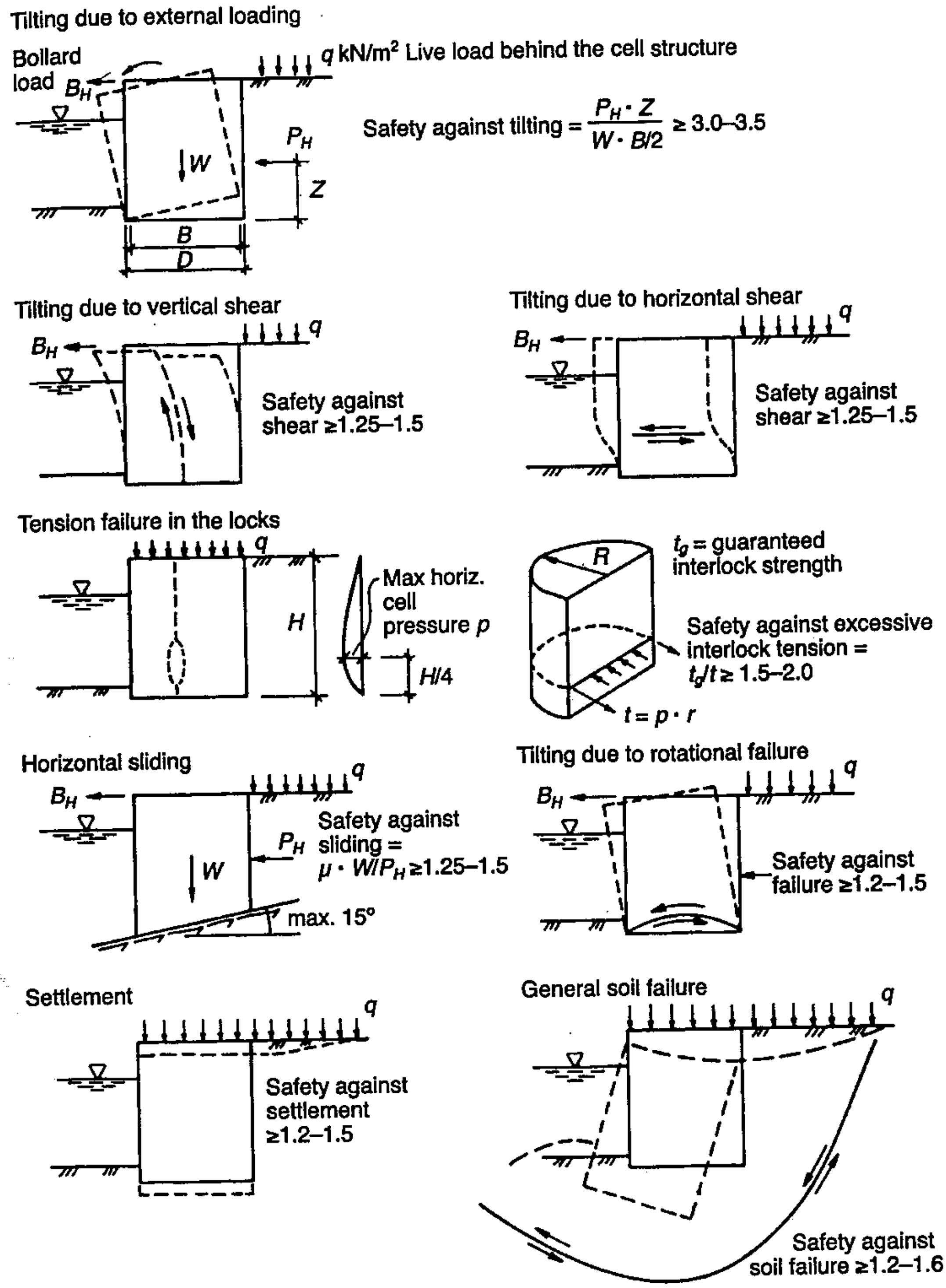


Fig. 9.12. Modes of cell failure

(k) The arched cell template is removed.
(l) The arched cell is filled.
(m) Points (i) through (l) are carried out for the second arched cell (between second and third main cells) when the filling of the third main cell is finished.

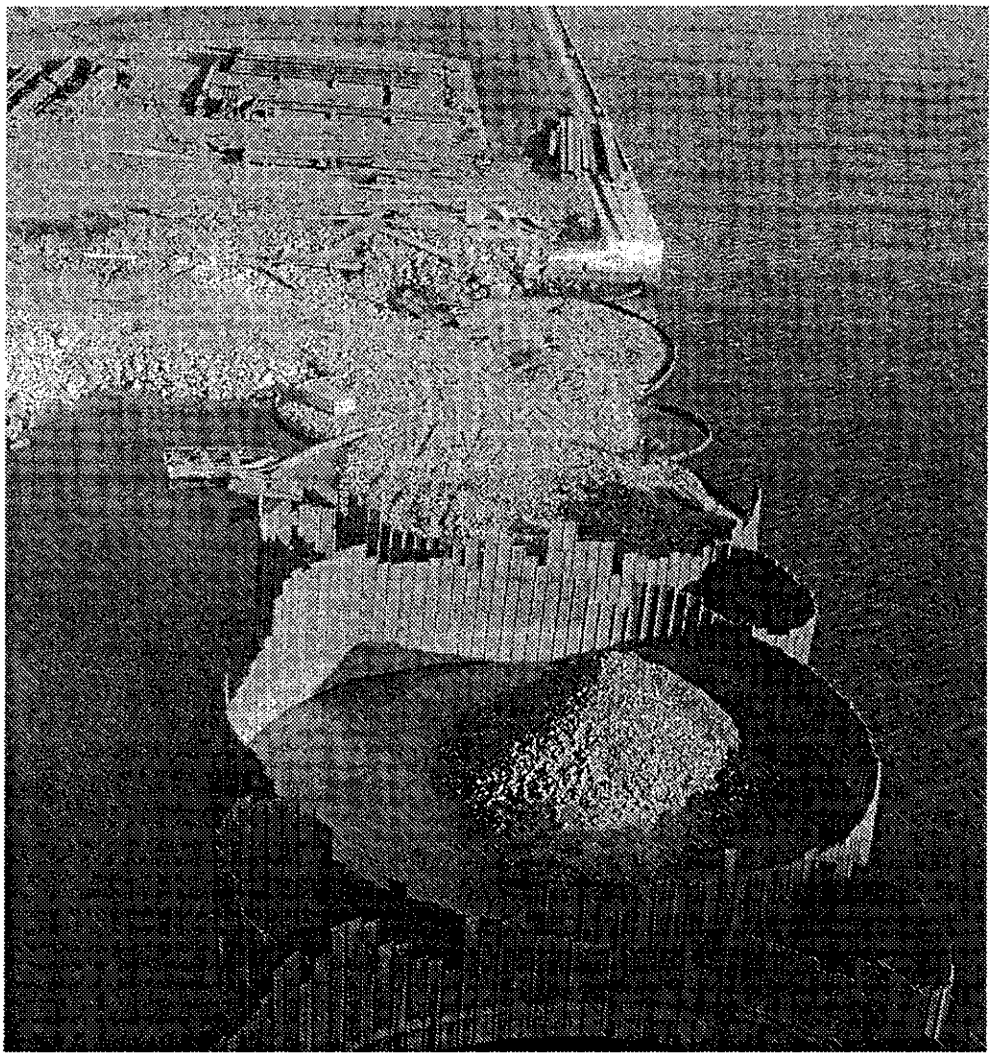


Fig. 9.13. Construction of a cell berth

(n) The cap (continuous concrete capping beam) is boarded, reinforced and concreted on top of the cells.
(o) Ordinary fill is placed behind the cap.
(p) Levelling of the fill and laying of the quay pavement, for instance asphalt.

It is extremely important in all types of cell construction that the template is adapted to the particular structure for which it shall be used. The template diameter must be correct so that the placing of the last sheet pile will just close the cell. The use of an internal or an external template is not critical, but internal templates are most often used. Figure 9.13 shows the construction of an extension of a cell berth and Fig. 9.14 shows the detail of the internal template and a cell before filling.

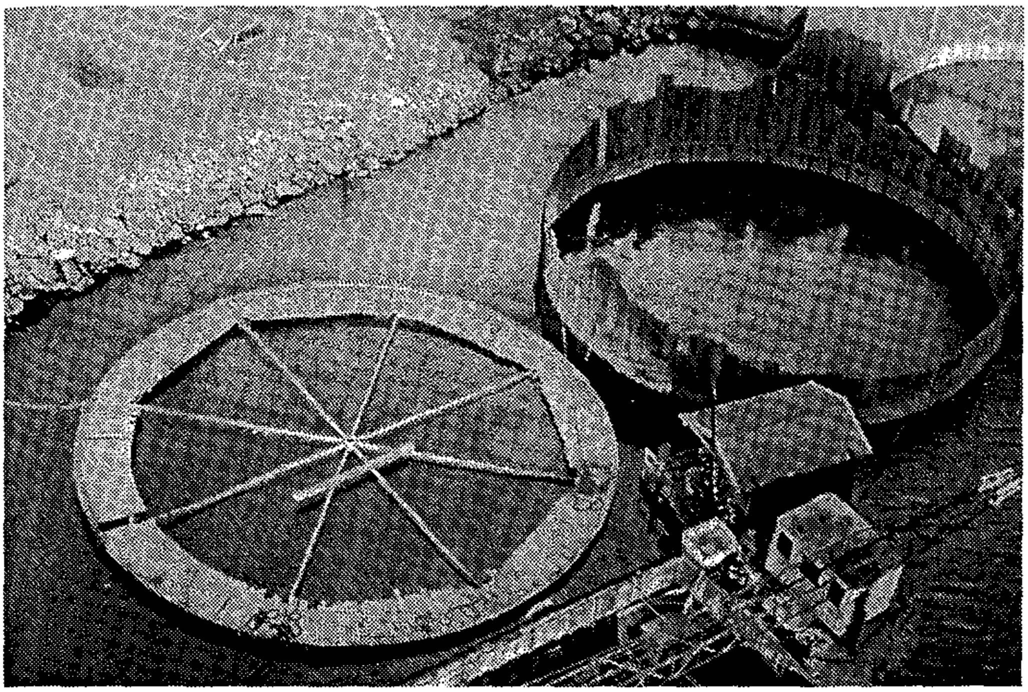


Fig. 9.14. Detail of the construction of a cell berth

The template must be sufficiently strong to resist any wind and wave pressure occurring at the site during construction. It must be able to absorb the loads and stresses transferred to it by way of the sheet piles when these are being placed and driven. Before these piles are firmly founded the wind and wave forces must be absorbed in full by the template. It should be noted that driving could not start before the cell is closed. The contractor must therefore plan carefully how the forces shall be absorbed without distortion or collapse of the template.

As far as possible preparation should be made to shorten the time needed for the placing of the sheet piles. The design engineer should also keep in mind that the smaller the cell diameter, the less construction time and forces from wind, waves and currents are to be expected. Thus the contractor and the engineer should ensure the correct placing and the smooth closing of the cell by a careful planning of the working procedure. The diameter of the filled cell will be 1–2 per cent greater than the theoretical diameter of the cell after the slack in the piling interlocks has been tightened due to the filling of the cell.

Crane and driving equipment must be adapted to suit the lengths of the sheet piles. It is quite usual to have pile lengths of 20–25 m. The driving must be carried out continuously along the whole circumference of the cell, in several stages, so that the penetration of a pile is not more than 50–70 cm compared to the neighbouring pile. The energy needed per blow is normally 6–10 kN m, obtained by use of a vibration hammer or double-acting air hammer giving 100 to 200 blows/min. A geotechnical engineer in each case should evaluate the driving criterion.

Splicing of sheet piles should be minimized and should not be allowed above elevation −3.0 m. The sheet pile ends should be cut at right angles. Only two splices in each sheet pile should be allowed. The vertical distance between splices in adjacent sheet piles should be a minimum 1.0 m. After threading and soft driving is completed and, before any filling takes place, the geometry of the cell should be controlled.

The geometry of the cell should comply with the following requirements:

(a) For all cells and half cells, the centre position should not deviate from the theoretical position by more than 200 mm.
(b) The ovality of the cell ($D_{max} - D_{min}$) should not exceed 500 mm for any of the cells.
(c) The inclination of any sheet pile in a radial or tangential direction should not exceed 40:1.
(d) If significant deviation from the required cell geometry is registered, provisions to correct the deviation should be made before the cell is filled at each stage of erection.

In soil containing sizeable stones it may be difficult to drive the sheet piles without damaging them. It is quite possible to place the cells directly on bare rock, but it is then necessary to secure a firm grip in the rock surface for the sheet piles and to pack around the base to prevent the washing out of the fill inside the cell or the sliding of the cell on the rock. As a rule of thumb, due to the sliding of the cell on rock surface, the slope of the rock surface should be less than 15°. If necessary, a horizontal shelf is provided by blasting into the rock surface to secure a safe founding of the cell.

For the filling of the cells sand, gravel or stone is used, giving only small specific settlement after filling and moderate soil pressure in the cells. In the case of filling with excavated rock, the size of blocks should be restricted to maximum 300 mm. The cell fill material should, in countries like Norway, during winter be free from ice and snow. If dredging is needed in front of the berth, it should be investigated whether the dredged material is inorganic and suitable for filling in the cells, for instance by suction dredging. The economy of cell berths, as compared with open columns or piled berths, depends very much on the cost of filling the cells with suitable material. In order to counteract the development of a possible hydrostatic head inside the cell, it is customary to make weep holes of size 6–10 cm around the cell circumferences. Care should be taken to not get any washout of the fill material through the weep holes.

Cell berths have the following advantages and disadvantages:

(a) Unlike the traditional sheet pile berths, which become unduly expensive for increasing water depth, the cell berths become more competitive the greater the water depth. As mentioned above, the weight of the steel/lin m of berth front is almost independent of the cell diameter. The minimum cell diameter is determined by the stability factor, and the allowable tensile load on the sheet pile locks determines the maximum diameter. Thus the latter determines the maximum water depth at the cell berth front.
(b) Diver work is only needed when the cell berth is founded directly on rock which requires the blasting of a horizontal shelf on the rock surface and possibly also in situ casting of a concrete beam around the cell, or dowelling the sheet piles into the rock. In other cases, only inspection by a diver is needed.
(c) Cell berths can be used for most types of soil condition provided that the load-bearing strength and stability of the ground are such that the load from a solid berth structure can be resisted.
(d) Cell structures are able to absorb considerable deformation, both vertical and horizontal, during construction. However, the sheet piles along the berth front must, in any case, be driven to a non-yielding stratum in order to secure a firm base for the capping beam. It is not unusual that the circular cell is slightly transformed into an oval during the filling up of the cell, but this is of no importance for the structure as such.
(e) The great load-bearing capacity of the cell berths is one of their most important advantages. The cell structures are very well adapted to resist heavy horizontal loads from the backfill, as well as from berthing and mooring forces. Therefore they can also be used in pier structures as the structural elements thus ensuring the transmission of horizontal loads down to the ground. In favourable ground conditions the load-bearing capacity regarding vertical live loads can be as much as 250 kN/m^2, while 30–50 kN/m^2 is usual for column berths. The cell berths are also able to resist great point loads, like wheel loads, etc., which usually create great difficulties in open berth structures.
(f) Compared to other types of berth structures, the cell berths require shorter construction periods.

(g) The risks of damage to the cells due to ship collision should always be considered. If the bow of a ship runs into the cell and penetrates it, the fill may run out of the cell and the stability of the structure may be greatly endangered. This applies in particular to pier head structures depending solely on the stability of the cells. Repair of such holes in the cell is often time-consuming and costly.

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10

Sheet pile wall structures

10.1 General

Like the gravity wall berths, the sheet pile wall berths can be sub-divided into the following three groups depending on how the load-bearing elements are designed:

(a) simple sheet pile wall berths
(b) solid platform berths
(c) semi-solid platform berths.

The sheet pile wall itself is unable to resist the horizontal loads from backfill, moving forces, etc. A part of the horizontal force is absorbed by passive earth pressure in front of the sheet piles through their fixation in the seabed, and tied anchor rods to an anchor structure behind the sheet pile wall will transmit a part.

In general, sheet pile walls are not suitable where the bedrock is found at such a high level that the loose seabed material left above it is insufficient to secure the fixation of the sheet piles. However, where the seabed is bare rock it is possible to fix the sheet piles to the rock by dowels or to provide an anchored concrete beam on the rock to prevent sliding of the piles from the horizontal forces. It must always be checked that the ground under the piles is able to resist the load and that the stability of the whole structure is secured.

Sheet pile materials can be wood, reinforced concrete or steel. In earlier times wood and reinforced concrete were often used in walls of modest heights, but nowadays reinforced concrete is usually only used in small or secondary structures, while wood is hardly ever used. The steel sheet piles are, nowadays the most widely used sheet wall elements in berth structures. Therefore only sheet piles made of steel are considered in the following.

The many manufacturers of deep-arch steel sheet piles produce a variety of profiles, but generally they can be divided into three main types:

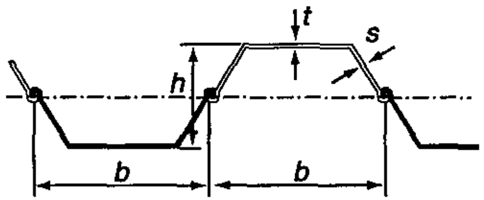
(a) The **U profile** sheet pile, where the locks are located at the neutral axis of the profiles, as in, for instance, the Larssen profiles. Characteristics are good corrosion resistance due to the biggest steel thickness lying on the outer part of the geometry and the ease of installing tie rods and swivelling attachments, even underwater.
(b) The **Z profile** sheet pile where the locks are located at the flange of the profiles, as in Krupp, Hoesch, Arcelor, Peiner profiles, etc. Main characteristics of the Z profile sheet pile are the continuous form of the web and the specific symmetrical location of the interlock on both sides of the neutral axis.
(c) The **H piles**, box piles and tubular piles which have either interlocking sheet pile elements or separate interlocks.

All the above profiles are produced in various sizes and with various moments of resistance and weights/m^2. Figures 10.1 and 10.2 give the main specifications for some of the most common deep-arch steel sheet pile profiles from Arcelor, Luxembourg. Also special sheet piles for use at wall corners, at transitions from one type of profile to another, etc. are produced. The steel sheet piles have a very great moment of resistance due to their weight, compared to sheet piles of reinforced concrete.

From Figs 10.1 and 10.2 the sheet pile bending moment capacity $M = W \times \sigma$ can be found.

For sheet piles having their locks at the neutral axis, like the U profiles, the useful moment of resistance must in some cases be reduced due to risk of sliding of the locks. In order to reduce such reduction, one can order from the manufacturer two and two sheet piles welded together, or the welding can be done at the site.

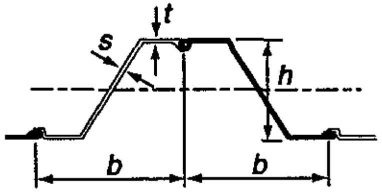
Steel pipe piles can be jointed together into a continuous retaining wall by connecting them with weld-on interlocking sections or into a combined wall comprised of conventional sheet piles as intermediate sections. Such pile walls and combined walls have enhanced resistance to vertical and horizontal loads compared with conventional sheet pile sections. The pipe piles in a retaining wall are usually open-ended but



Section	b (mm)	h (mm)	t (mm)	s (mm)	Coating area* m^2/m^2 wall	Steel section cm^2/m	Mass kg/m single pile	Mass kg/m^2 wall	Section modulus** cm^3/m wall	Moment of inertia** cm^4/m wall	Radius of gyration cm/m wall	Bending moment capacity at yield point S 240 GP	S 355 GP kNm/m	S 430 GP
AU 14	750	468	10.0	8.3	2.54	132	77.9	104	1410	28 710	14.73	338	501	606
AU 16	750	411	11.5	9.3	2.54	147	86.3	115	1600	32 850	14.98	384	568	688
AU 17	750	412	12.0	9.7	2.54	151	89.0	119	1665	34 270	15.05	400	591	716
AU 18	750	441	10.5	9.1	2.65	150	88.5	118	1780	39 300	16.17	427	632	765
AU 20	750	444	12.0	10.0	2.65	165	96.9	129	2000	44 440	16.43	480	710	860
AU 21	750	445	12.5	10.3	2.65	169	99.7	133	2075	46 180	16.52	498	737	892
AU 23	750	447	13.0	9.5	2.71	173	102.1	136	2270	50 700	17.10	545	806	976
AU 25	750	450	14.5	10.2	2.71	188	110.4	147	2500	56 240	17.32	600	888	1075
AU 26	750	451	15.0	10.5	2.71	192	113.2	151	2550	58 140	17.39	619	915	1109
PU 6	600	226	7.5	6.4	2.37	97	45.6	76	600	6780	8.37	144	213	258
PU 8	600	280	8.0	8.0	2.50	116	54.5	91	830	11 620	10.02	199	295	357
PU 12	600	360	9.8	9.0	2.64	140	66.1	110	1200	21 600	12.41	288	426	516
PU 12 10/10	600	360	10.0	10.0	2.64	148	69.6	116	1255	22 580	12.36	301	446	540
PU 16	600	380	12.0	9.0	2.75	159	74.7	124	1600	30 400	13.85	384	568	688
PU 20	600	430	12.4	10.0	2.91	179	84.3	140	2000	43 000	15.50	480	710	860
PU 25	600	452	14.2	10.0	3.03	199	93.6	156	2500	56 490	16.86	600	888	1075
PU 32	600	452	19.5	11.0	3.03	242	114.1	190	3200	72 320	17.28	768	1136	1376
L 25	500	340	12.3	9.0	2.92	177	69.7	139	1600	27 200	12.38	384	568	688
L 35	500	400	14.1	10.0	3.04	201	78.9	158	2000	40 010	14.11	480	710	860
L 45	500	440	15.5	10.0	3.22	219	86.2	172	2500	55 010	15.83	600	888	1075
JSP 3	400	250	13.0		2.98	191	60.0	150	1340	16 800	9.38	322	476	576

Notes: * 2 sides, inside of interlocks excluded; ** Shear transfer in the interlock must be assured to guarantee the given values

Fig. 10.1. U steel sheet pile profiles from Arcelor, Luxembourg



Section	b (mm)	h (mm)	t (mm)	s (mm)	Coating area*	Steel section	Mass		Section modulus**	Moment of inertia**	Radius of gyration	Bending moment capacity at yield point		
					m²/m² wall	cm²/m	kg/m single pile	kg/m² wall	cm³/m wall	cm⁴/m wall	cm/m wall	S 240 GP	S 355 GP kNm/m	S 430 GP
AZ 12	670	302	8.5	8.5	2.45	126	66.1	99	1200	18 140	12.02	288	426	516
AZ 13	670	303	9.5	9.5	2.45	137	72.0	107	1300	19 700	11.99	312	462	559
AZ 14	670	304	10.5	10.5	2.45	149	78.3	117	1400	21 300	11.96	336	497	602
AZ 17	630	379	8.5	8.5	2.70	138	68.4	109	1665	31 580	15.12	400	591	716
AZ 18	630	380	9.5	9.5	2.70	150	74.4	118	1800	34 200	15.07	432	639	774
AZ 19	630	381	10.5	10.5	2.70	164	81.0	129	1940	36 980	15.03	466	689	834
AZ 25	630	426	12.0	11.2	2.82	185	91.5	145	2455	52 250	16.80	589	872	1056
AZ 26	630	427	13.0	12.2	2.82	198	97.8	155	2600	55 510	16.75	624	923	1118
AZ 28	630	428	14.0	13.2	2.82	211	104.4	166	2755	58 940	16.71	661	978	1185
AZ 34	630	459	17.0	13.0	2.93	234	115.5	183	3430	78 700	18.36	823	1218	1475
AZ 36	630	460	18.0	14.0	2.93	247	122.2	194	3600	82 800	18.30	864	1278	1548
AZ 38	630	461	19.0	15.0	2.93	261	129.1	205	3780	87 080	18.26	907	1342	1625
AZ 46	580	481	18.0	14.0	3.26	291	132.6	229	4595	110 450	19.48	1103	1631	1976
AZ 48	580	482	19.0	15.0	3.26	307	139.6	241	4800	115 670	19.43	1152	1704	2064
AZ 50	580	483	20.0	16.0	3.26	322	146.7	253	5015	121 060	19.38	1204	1780	2156

Note: * 2 sides, inside of interlocks excluded

Fig. 10.2. Z steel sheet pile profiles from Arcelor, Luxembourg

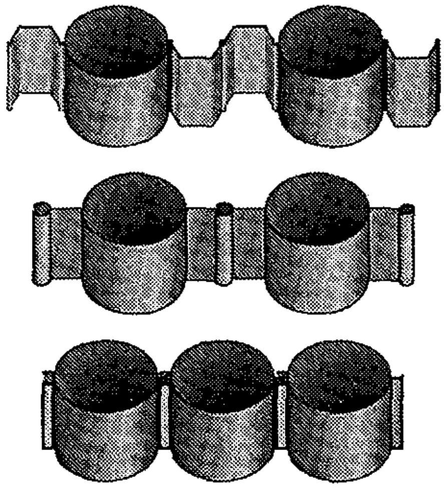


Fig. 10.3. Steel tubes as sheet pile wall

can be equipped, if necessary, with pile shoes. An advantage of steel pipe pile walls is that they can be driven into most hard-to-penetrate soils. Figure 10.3 shows steel tubes used as sheet pile wall.

The steel grades employed for steel sheet piling should be in accordance with DIN EN 10 248 as shown in Table 10.1 or equivalent.

Table 10.1. Steel grades according to DIN EN 10 248

Steel grade*	Minimum tensile strength	Minimum yield point	Minimum elongation	Permissible stresses**		
				Loading class 1	Loading class 2	Loading class 3
S 240 GP	340	240	26	160	184	208
S 270 GP	410	270	24	180	207	234
S 355 GP	480	355	22	240	276	312
S 390 GP	490	390	20	260	299	338
S 430 GP	510	430	19	290	334	559
Steel grade employed for HOESCH trench sheet piles and lightweight sections in accordance with DIN EN 10 249						
S 275 JRC***	410	275	22	185	213	241

* Steel grade designation by DIN EN 10 248: S 240 GP (40 A), S 270 GP (43 A), S 355 GP (50 A)
** In case of compression and bending stresses for the analysis of stability, reduced permissible stresses apply (see Recommendation R 20 of the *Recommendations of the Committee for Waterfront Structures*, EAU 1990, EAU 1996)
*** Steel grade designation to DIN EN 10 249: S 275 JRC (43 B)

10.2 Driving of steel sheet piles

The driving of steel sheet piles of corrugated steel sheet piling of Z and/or U profiles should always be driven in pairs. Driving of single sheet piles should, if possible, be avoided. Driving with triple or quadruple piles may have technical and economical advantages in specific cases.

For driving, slow-stroke drop hammers, diesel hammers, hydraulic hammers and rapid-stroke hammers may be used. The efficiency of the sheet pile driving generally increases when the ratio of driving hammer weight to the weight of the driving element including the pile cap is increased. For free drop hammers, hydraulic hammers and single-acting steam hammer a ratio of hammer weight to the weight of the driving element of 1:1 is preferred. For a rapid-stroke hammer a ratio of 1:4 is preferred. Slow-stroke heavy hammers are recommended in cohesive soils, while in non-cohesive soils rapid-stroke hammers are recommended.

During driving, the Z-shaped sheet piles have a tendency to lean backwards due to the driving direction, while the U-shaped sheet piles tend to lean forward in the driving direction.

The EAU 1996 recommends that the following driving deviations and tolerances should be included in the calculations at the planning stage:

(a) 1.0 per cent for normal soil conditions and driving on land
(b) 1.5 per cent of the driving depth for driving on water
(c) 2.0 per cent of the driving depth with difficult subsoil.

With increasing driving depth of the sheet piles, the deviation from the vertical will increase.

10.3 Simple sheet pile wall berths

When water depth at the berth front is moderate and the soil conditions are favourable, a simple sheet pile wall, as shown in Fig. 10.4, can be an economically good solution. The sheet piles are driven down to sufficient depth, one after the other. The horizontal anchor force is transmitted from the wale (anchoring beam) by way of a tie rod to a retaining plate or other type of anchorage. It is advantageous if the wall can be driven and anchored first, and the dredging, to the prescribed depth, in front of the wall can be done afterwards.

The most economical free wall height of a simple sheet pile wall berth with a simple anchoring system varies from about 7–10 m, depending

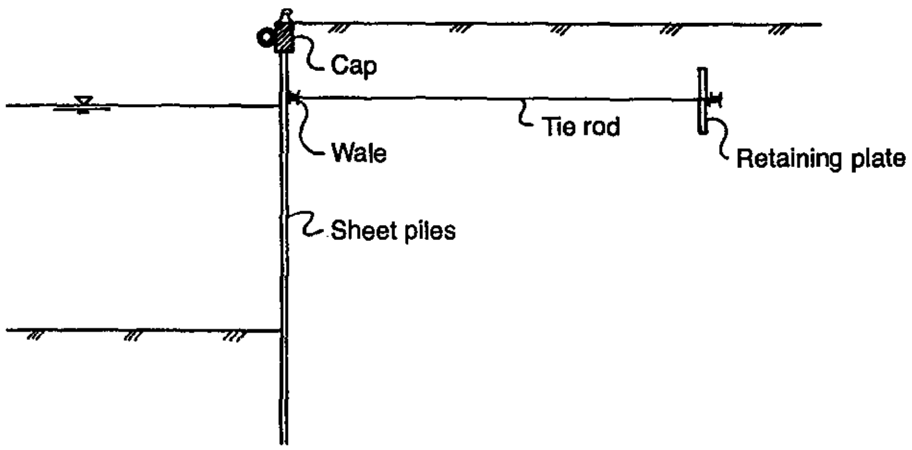


Fig. 10.4. A simple sheet pile wall

on the soil conditions, price of steel, driving equipment and the useful load on the quay deck.

The berthing loads normal to the berth structure are transmitted to the earth behind the structure by the fender system either to the capping beam or to the sheet wall itself. The horizontal loadings in the longitudinal direction from friction between the berthing ship and the fender system are taken by the sheet pile wall itself.

For detailed geotechnical design calculation of the steel sheet pile structures, see 'Further reading' at the end of this chapter.

The maximum horizontal loading on the wall arises when an extremely low water level in front of the berth and extremely high groundwater level behind the wall occur together with maximum live load and mooring forces on the structure. Low-water level in front of the berth, due to low tide, wind, air pressure, etc., cannot be avoided, but a high hydrostatic pressure behind the wall can normally be avoided by using coarse-graded fill combined with a special drainage system to prevent excessive inside water pressure. Still, in the design of the sheet piles provision should be made for a periodical difference of water levels on the two sides of the wall.

The active soil pressure on the wall determines the cross-sectional dimensions of the sheet piles. The passive soil pressure determines the support of the wall at its lower end. The horizontal tie rod tension has to be absorbed either by the passive soil pressure in front of the retaining plate, as illustrated in Fig. 10.4, or by friction between a friction plate and the soil. The sheet pile and the wale are designed primarily to resist bending stresses, while the tie rod cross-section is determined by the maximum tensile stress it has to resist and also by its length.

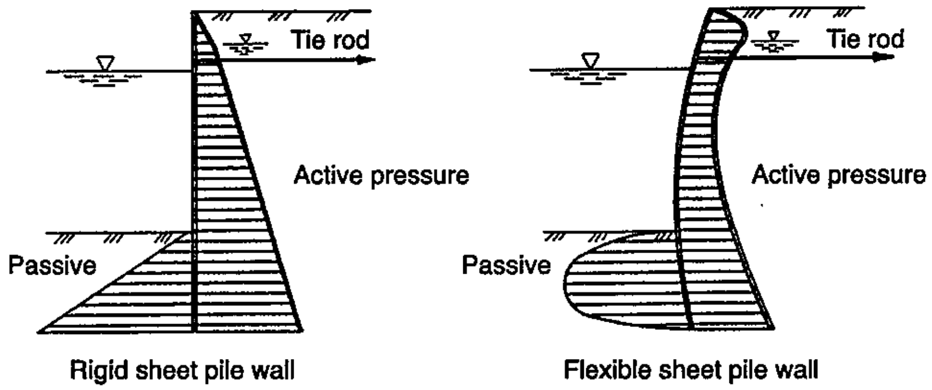


Fig. 10.5. Effects of the flexibility of anchored sheet pile wall

Thus the dimensions of the structural elements of the wall are determined by the soil conditions, the water-level variations, the duration of the construction work, method of filling behind the wall, filling period, etc. Taken together, these factors imply a rather complicated design pattern, in which the soil mechanics aspects must be given particular attention. Figure 10.5 illustrates the distribution of the active and passive pressure on single-anchored rigid and flexible sheet pile retaining walls.

The wall is finished on top by the installation of a special steel beam or by the casting of a reinforced concrete capping beam (cap). The latter is usually made wide enough for the installation of bollards and a quay-front kerb on top, and high enough for the installation of fenders on the front. In major berths the bollards are usually anchored through their own tie rods to the retaining plates. If the berth has cranes on rails, the capping beam is often utilized as support also for the outer rail. Account must then be taken of the necessary free clearance between crane and berthing ships and possible settlement of the wall.

Expansion joints in the capping concrete beam are placed at intervals of between 15–30 m depending on the structure and the amount of reinforcement in the capping beam to absorption of the increased tensile forces in the longitudinal direction of the capping beam.

Different methods and principles are applied in the design of anchoring for sheet pile walls. Some of the most used anchoring systems are described in the following. In principle these systems can also be used for the anchoring of other types of berth structure. Within harbour construction it generally applies that underwater work should, if possible, be avoided and that anchorage or other work

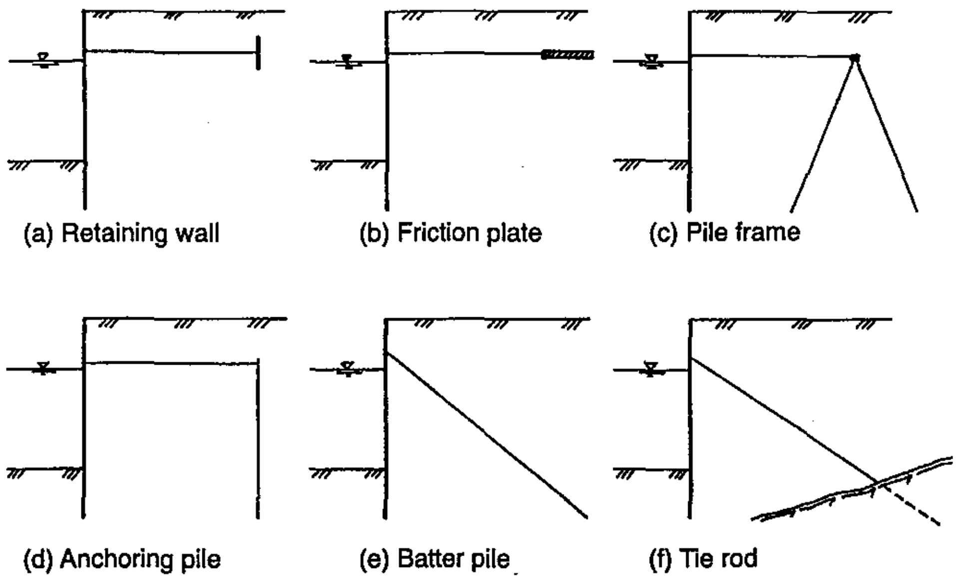


Fig. 10.6. Anchoring

should be achieved in the simplest possible way. This implies that the anchor points and the tie rods should be placed as close to the tidal zone as possible. The bending moment in the sheet piles will then be moderate and the underwater work could be avoided or made easier. Different ways of designing sheet pile wall anchoring, shown in Fig. 10.6, are as follows:

(a) Retaining wall supported by passive soil pressure in front of the wall. The retaining wall must in theory be pulled some small distance in the direction of the sheet pile wall before the passive soil pressure is fully mobilized. The soil in front of the retaining wall should therefore be thoroughly compacted to minimize this movement before the tie rod is fixed.
(b) Retaining plate or friction plate transmitting the force only by way of friction to the soil under the plate. This takes place when the width of the plate is bigger than the thickness of the soil layer above the plate. The force is then transmitted to the underlying soil without any movement of the plate.
(c) The force is transmitted to pile frames, each consisting of one tension pile and one compression pile. The piles can be rather long and thick in order to provide the necessary contact area to the soil. The piles should, if possible, be positioned behind the active soil wedge to allow frictional resistance to be developed along the full length of the piles.

(d) The force is absorbed through the bending of anchoring piles. This is a convenient method when the distance from the berth front to the anchoring point is limited and the force is moderate.
(e) Direct anchoring by way of batter friction piles behind the wall absorbs the force. The batter friction piles will be rather long and thick in order to provide the necessary contact friction area. This method sets up vertical downward directed forces in the sheet piles, which can cause settlements of the wall.
(f) The wall is anchored by way of tie rods down to firm rock, or to anchors in the soil. This is a suitable method of temporary anchoring during repair of the wale, etc., and where none of the above methods can be utilized.

The tie rods should be designed to accommodate any settlement of the ground under the tie rods. If noticeable settlements of the fill behind the wall are expected, the connection between sheet piles and tie rod should be formed as a hinge, and wooden poles, to prevent damage due to settlement in the filling, should support the rods. Usually the tie rods are placed just above the upper level of the tidal zone. When there are also great tidal variations a lower tie rod can be installed just above the lower level of the tidal zone. Thus the bending moments in the wall can be considerably reduced. Figure 10.7 shows details of a steel sheet pile wall, tie rod and retaining wall.

The tie rods are usually made of round bars of diameter between 5–10 cm and quality St. 37 or St. 52. Frequently-used types of end fastening and splicing of tie rods are shown in Fig. 10.7. Very often the tie rod ends are jolted so that their diameter at the bottom of the thread is the same as the diameter of the unthreaded rod. Due to difficulties in designing the tie rods exactly according to the tensile force, the rods are usually over-designed by 50–100 per cent. To protect the tie anchor system from additional loading by the weight of the backfill, the tie rods should be placed with negative sag of approximately 5–15 cm, depending on the length of the tie rod, to compensate for the settlement of the backfill. The tie rod with its connection should have a safety factor of at least 2 under all loading conditions. Joints, in which stress concentrations may occur, are also over designed by 50–100 per cent. This philosophy should be adopted also in the design of tie rods made of reinforced concrete.

The consequences of a possible failure of the anchorage should always be taken into account. Buckling or overloading of the sheet piles will usually not be as serious as an anchorage failure.

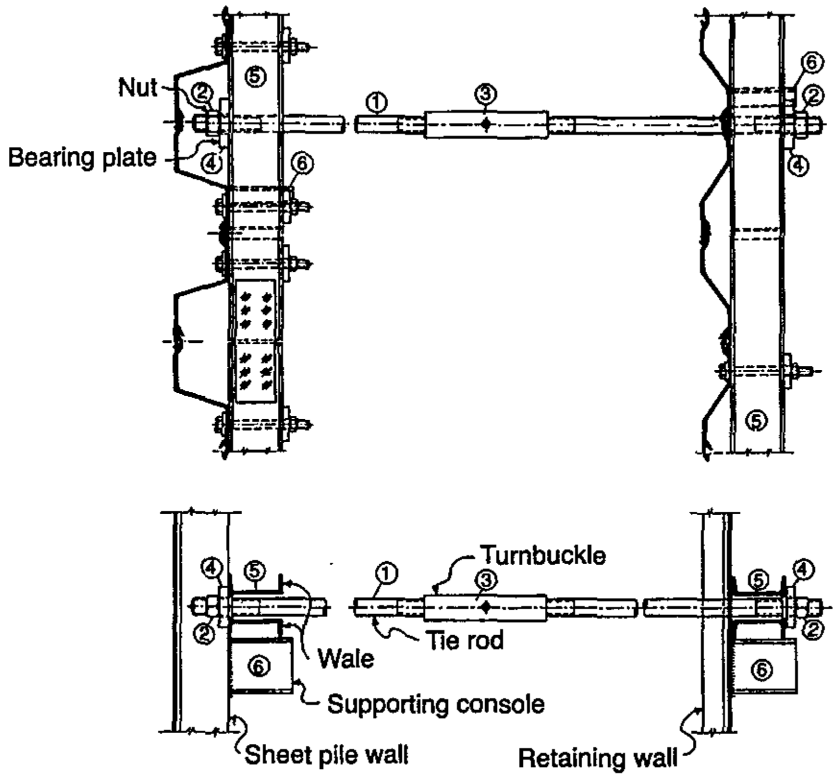


Fig. 10.7. Details of steel sheet wall anchoring

Steel tie rods are also normally protected against corrosion when they are located above the tidal zone. Most frequently a bituminous material is used and the tie rods are wrapped in canvas or similar and embedded in sand. If concrete is used as the protective material it must be reinforced against shrinkage cracks.

To obtain an estimate of the sizes of the forces and dimensions in simple steel pile wall berths for different live loads, diagrams of the type shown in Fig. 10.8 may be used. The diagrams are based on the assumption that the soil at both sides of the sheet pile wall consists of dense, sharp-grained sand or gravel. The angle of internal friction, φ, therefore is estimated to be 38°. The diagram should not be used for weaker soils, i.e. soils of $\varphi < 36°$. The main assumptions for the calculations are shown in the figure and the simplified diagram may be used for rough estimations.

The diagrams in Fig. 10.8 are calculated according to Publication 16 of The Norwegian Geotechnical Institute. The diagrams are based on classical earth pressure theories. As an illustration, the length of sheet pile, design moment, anchor force, length of tie rod and length of friction slab are shown for a sheet pile wall with water depth of 8 m and live load of 50 kN/m^2. In recent years several data programs

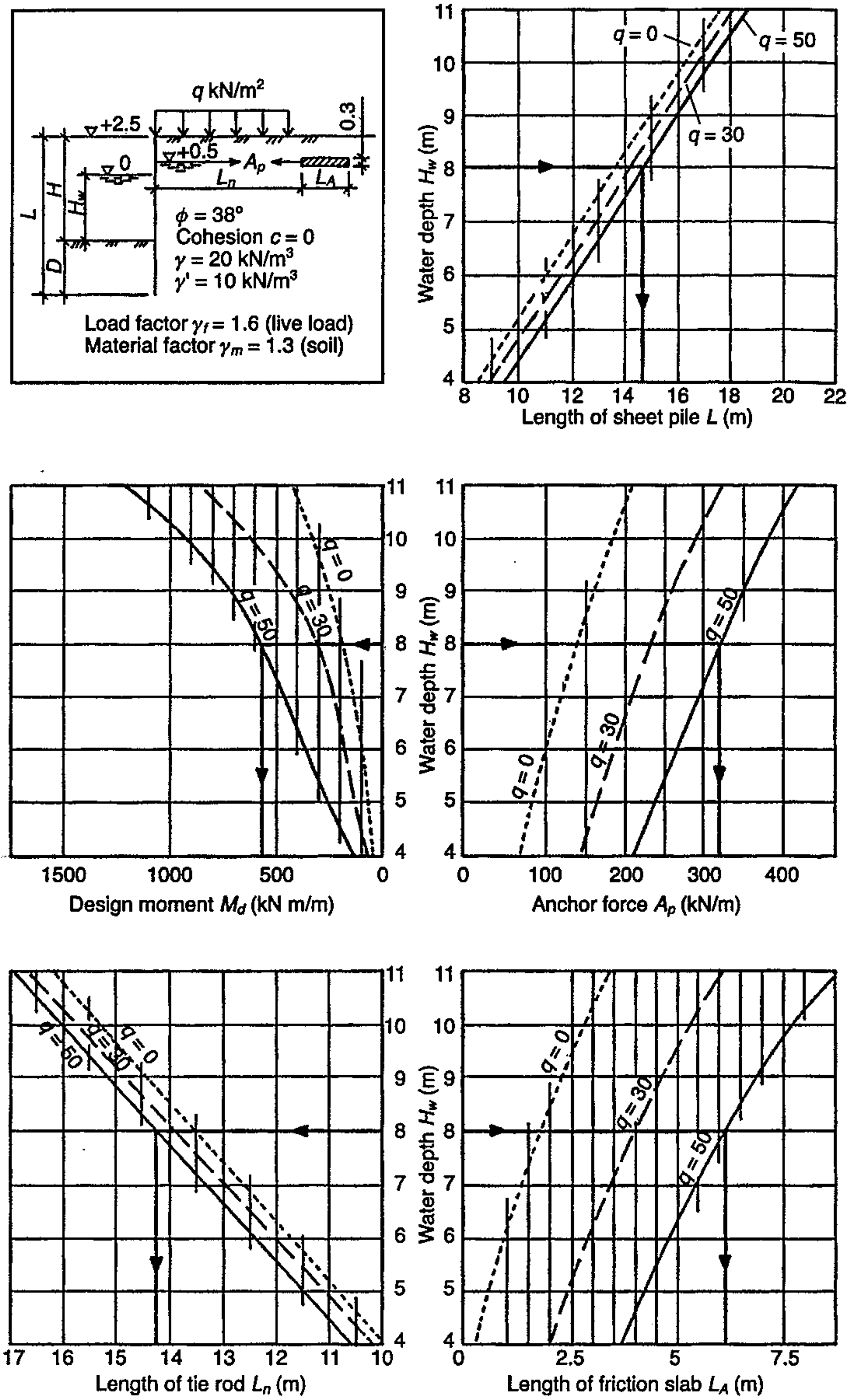


Fig. 10.8. Estimate diagram for steel sheet piles

have been developed accounting for several other factors like, for instance, the rigidity of wall and anchors.

10.4 Solid platform berths

When the height of the berth front exceeds about 8–10 m, the simple sheet pile wall structure will normally not be the most economical solution. Better solutions can be obtained by using one or more of the following adaptations:

(a) Tie rods are placed at two different levels. This can also be done where the tidal variations are great.
(b) Use of special sheet pile profiles designed to resist great bending moments.
(c) A relieving plate, placed on the wall and on piles behind the wall, transmits useful loads and the weight of the fill on top as axial loads to the wall and the piles, and reduces the horizontal load acting on the wall. This type of structure is called the solid platform type of berth, as shown in Fig. 10.9.

Solid platform berths are probably most interesting from an economic point of view in cases where the live load and/or the quay-front height are relatively high. Also where the berth-front height is relatively small, this type of structure can be economically justified provided very great live loads are acting on the berth deck. This type of structure has also been used in cases where the utilization of sheet piles with small moments of resistance has been pursued.

Horizontal forces are absorbed either by batter piles under the relief plate or by an anchorage further back. As mentioned under anchoring of simple sheet pile walls, a vertical retaining wall must be pulled a small

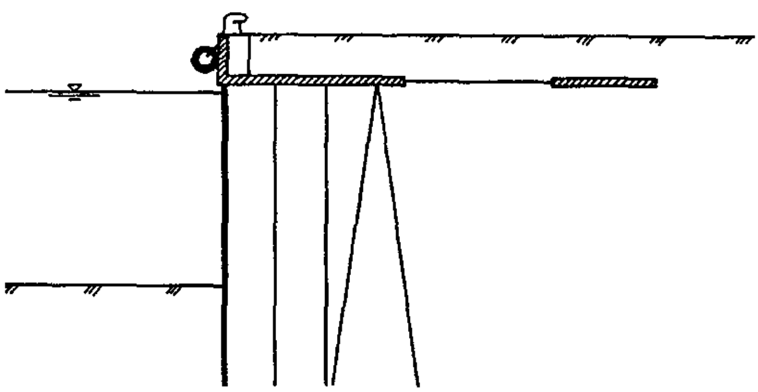


Fig. 10.9. Solid platform berth

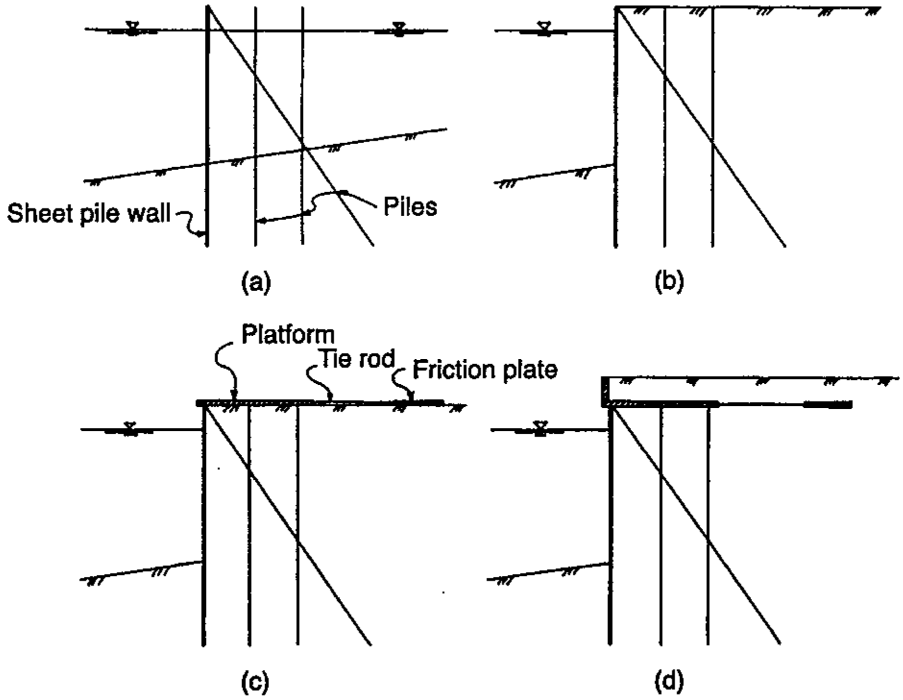


Fig. 10.10. Offshore construction

amount in the direction of the sheet pile wall before the passive soil pressure is fully mobilized, while a friction plate transmits the force directly to the underlying soil. This implies that batter piles and retaining plates can be utilized simultaneously.

The absorption of horizontal load by a combination of batter piles and retaining wall is not equally simple. In this case special precautions should be taken to make the piles and the wall act together, such as stretching of the tie rods. The choice of anchoring system depends on the method of construction to be utilized. There are principally two methods of constructing these berths:

Offshore: the berth is built independently in the water and the shore connection is filled afterwards. Figure 10.10 shows this procedure in principle. It is of decisive importance that the sheet pile wall is sufficiently anchored against soil pressure during the construction. When using the working sequence indicated in the figure, the platform can be cast directly on the filled area. Usually batter piles absorb the horizontal forces and/or retaining plate, as shown in the figure.

Onshore: the berth is built on shore and the basin in front of the berth is dredged after the installation of the anchoring and the filling on top, as shown in Fig. 10.11 and in Figs 10.12, 10.13 and 10.14. Batter piles are not needed in this case.

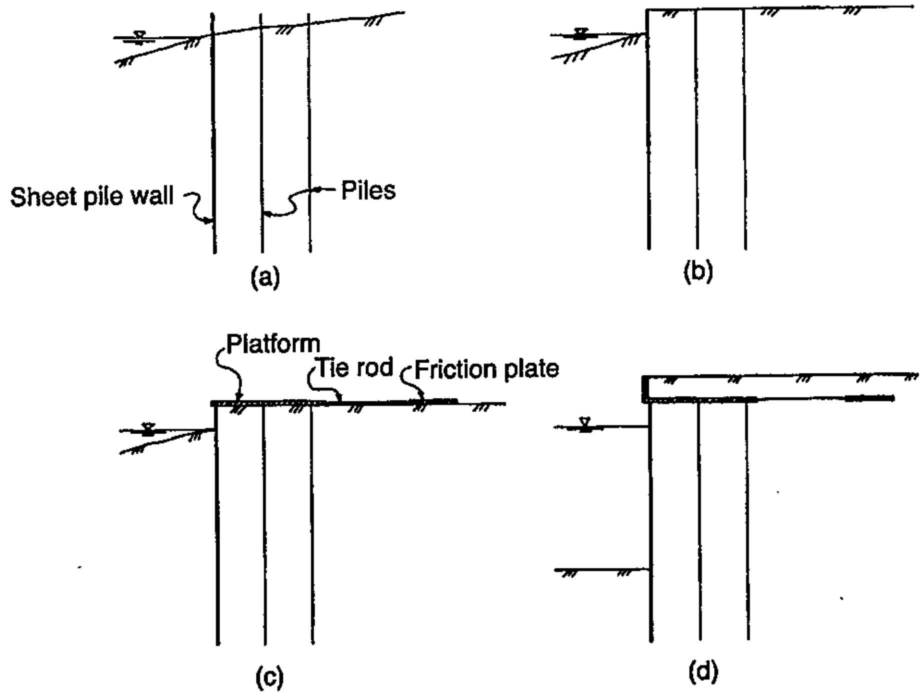


Fig. 10.11. Onshore construction

If one wants to avoid the construction of a retaining or friction plate, the structure can be designed as shown in Fig. 10.15. The platform is supported by the sheet pile wall and the pile frame and acts as an anchorage between the wall and the frame. In order to get the biggest possible stabilizing load on the pile frame and to reduce the

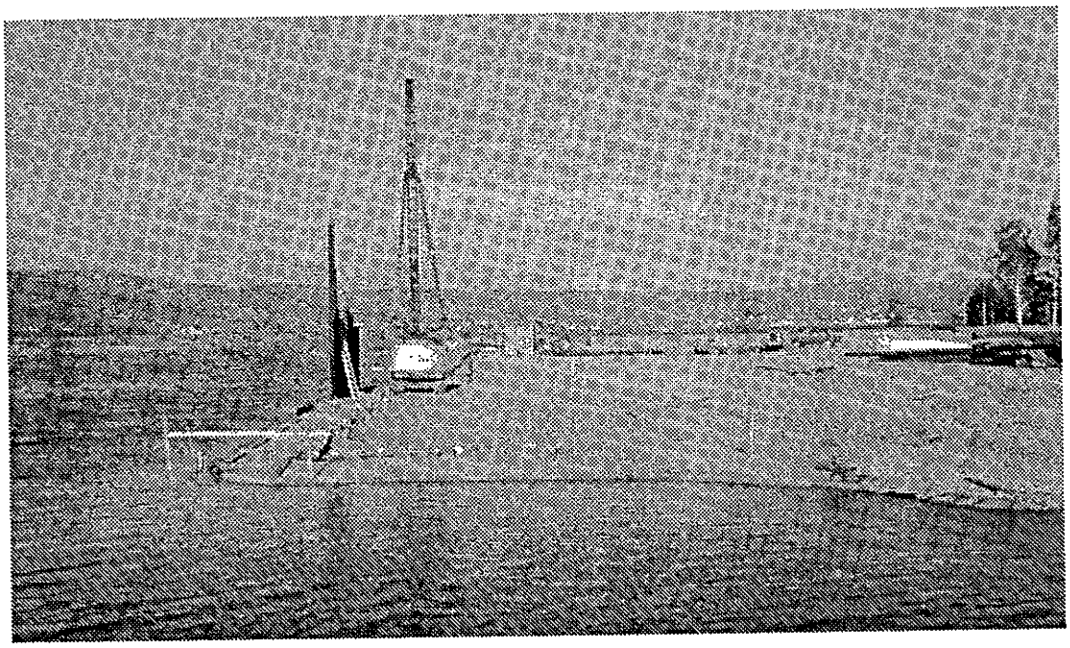


Fig. 10.12. Onshore construction phases (a) and (b)

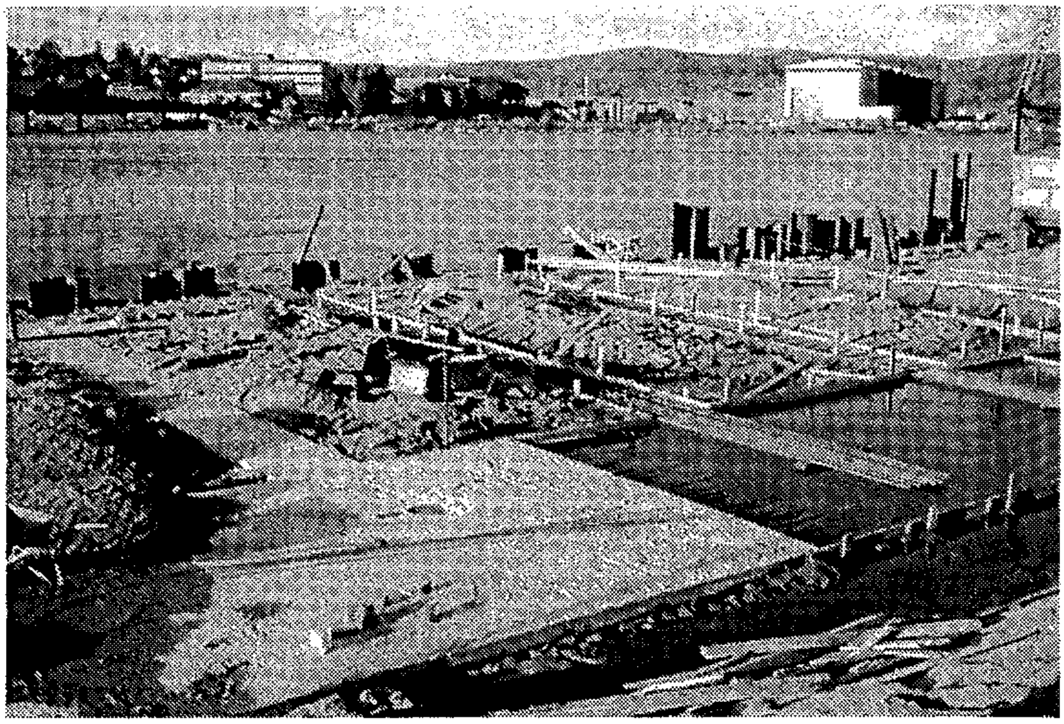


Fig. 10.13. Onshore construction, phase (c). The construction of tie rods and friction slab

bending moment in the platform span, the platform ought to form a cantilever to the inner side of the pile frame.

Neither in the offshore nor in the onshore method can any friction occurring between the platform and the underlying soil be assumed

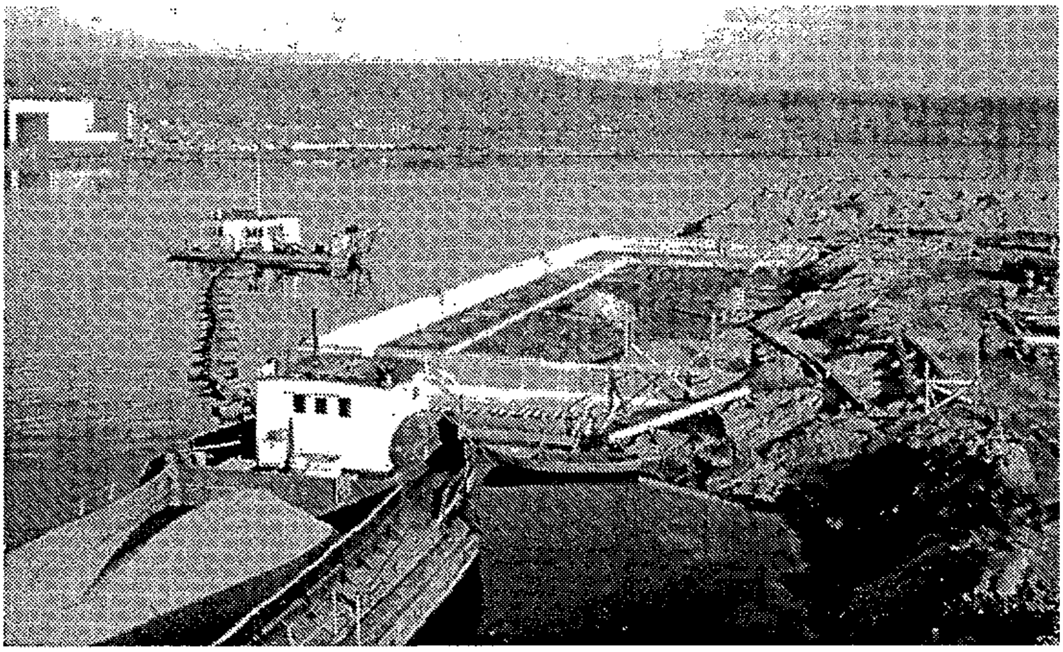


Fig. 10.14. Onshore construction, phase (d). The backfilling of sand

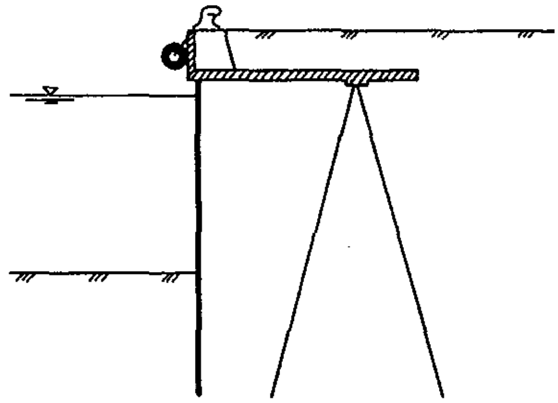


Fig. 10.15. Solid platform berth

because settlements under the platform will arise as time goes by. The platform and the retaining plate should be placed at such a high level that they can be constructed above water. Both wooden and concrete piles can be used under the platform. Wooden piles must be protected against rotting, and also against wood borers (sea worms) because settlements can take place underneath the platform.

The economically most favourable berth-front height for this type of berth is about 14–18 m, depending on the geotechnical conditions on site and the cost of sheet piles and driving.

10.5 Semi-solid platform berth

If a greater quay-front height than about 14–18 m is needed, the type of berth structure shown in Fig. 10.16 may prove to be suitable. In principle it is very much like the solid platform type, but the soil pressure against the sheet pile wall is further reduced by omitting some of the fill under the platform, as shown in the figure. This implies that a formwork has

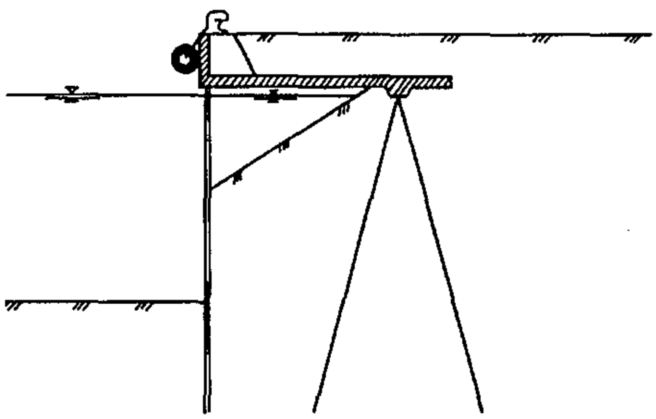


Fig. 10.16. Semi-solid platform berth

to be made for the platform, instead of casting it directly on the soil, as for the solid quays. To facilitate in situ work, the platform can be made of precast, reinforced concrete elements. Many such elements, of more or less sophisticated designs, have been introduced.

The main advantage of this semi-solid type, as compared with the solid type, is that the berth-front height can be made substantially higher. The disadvantages are that untreated wooden piles cannot be used, that the piles must have greater cross-sectional dimensions due to the risk of buckling and that corrosion may occur on both sides of the wall.

The semi-solid platform berth may be characterized as a transition from the solid berth structures to the open column and lamella berth structures.

10.6 Drainage of the steel sheet piles

The best form of drainage behind a steel sheet pile wall, to avoid extra pressure behind the wall during low tide or after a heavy and long rain period, is to have graded coarse friction materials approximately 1 m behind the wall, well below groundwater level. Effective drainage is only possible in non-cohesive soils.

In addition, one should install, if possible, weep holes below mean water level. The weep holes should be located above low-water level to allow maintenance, and in such a way that they cannot be damaged by ships or floating debris. The weep holes should be sealed with a suitable filter to prevent loss of fill materials. Weep holes should not be used if there is danger of heavy growth of barnacles. The design of weep holes should take into consideration the possibility of blockage due to marine growth.

References and further reading

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11

Open berth structures

11.1 General

The open berth structures constitute, with their berth platforms, a prolongation over the slope from the top of the filled area out to the berth front. In this chapter only berth structures in reinforced concrete, or platforms in reinforced concrete founded on concrete-filled tubular steel piles, will be described. Open berth structures built of wooden materials will not be described, but construction principles, load-bearing capacity, etc. are largely the same as for structures in reinforced concrete.

In the same way as for solid berth structures, the open berth structure can also be divided into the following main types, depending on the principles according to which the front wall and the platform are designed to resist the loading, so that the berth structures have the necessary stability.

(a) **Column or pile berths**: the berth platform and the berth front wall are founded on either columns or piles, or a combination of both, which do not have a satisfactory stability against external forces. Therefore, the berth structure must be anchored, for instance by a friction plate in the filling. The structure must then be built simultaneously with the filling or preferably after the fill has been established.

(b) **Lamella berths**: the berth platform and the berth front are founded on vertical lamellas, which provide the loaded berth structure with a satisfactory stability. The berth structures are stable enough in themselves to resist loads from ships, live loads, possible pressure from fill at the rear of the structure, etc. without anchoring of

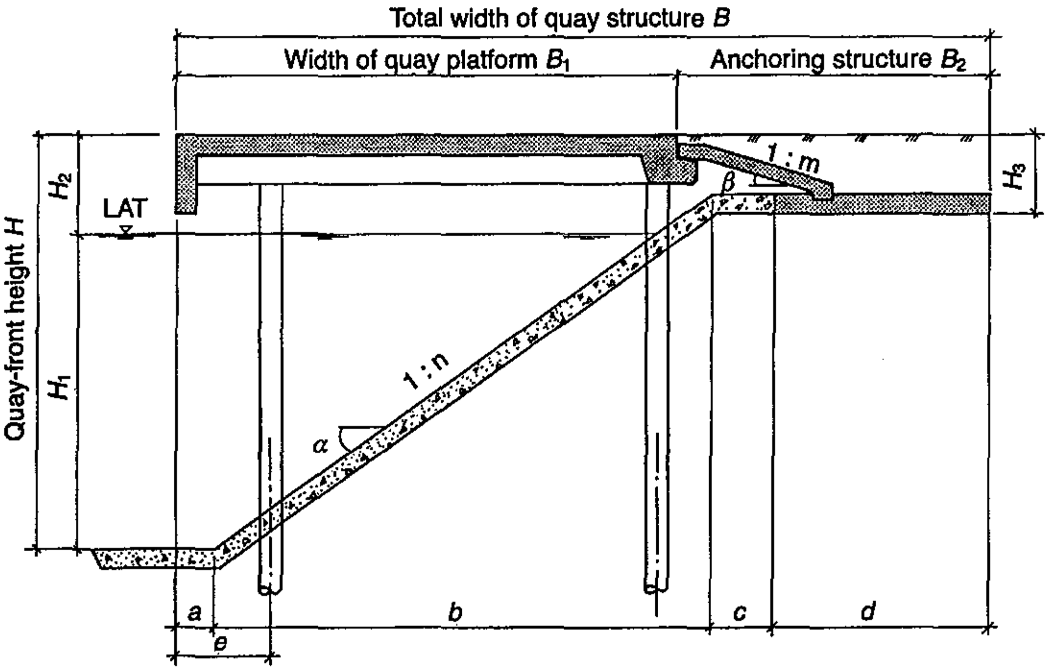


Fig. 11.1. Characteristics of open berth structure

the structure. In the same way as for gravity-wall structures, one can build the lamella berth itself first, and then fill behind close up to the structure.

Very often the two types are combined. For instance, in a pier or jetty type of berth structure the shore base is anchored by a retaining wall while, at the head of the jetty, the horizontal loading is resisted by lamellas. The berth platform between the shore base and the head of the structure is founded on columns and/or piles.

The open berth structure when compared to the massive berth structure is the most suitable in the following circumstances:

(a) The seabed is too weak to carry a massive berth structure.
(b) The ground condition below the seabed is suitable for bearing piles.
(c) Large water depth.
(d) The need to minimize the hydraulic regime.
(e) Difficulties in getting suitable backfill materials for a retaining wall berth structure.

Figure 11.1 shows the cross-section of a pile berth anchored at the rear embankment and indicates the main characteristics of open berth structures. This method of accommodating the horizontal forces by use of an anchor and friction slab is a typical Norwegian design.

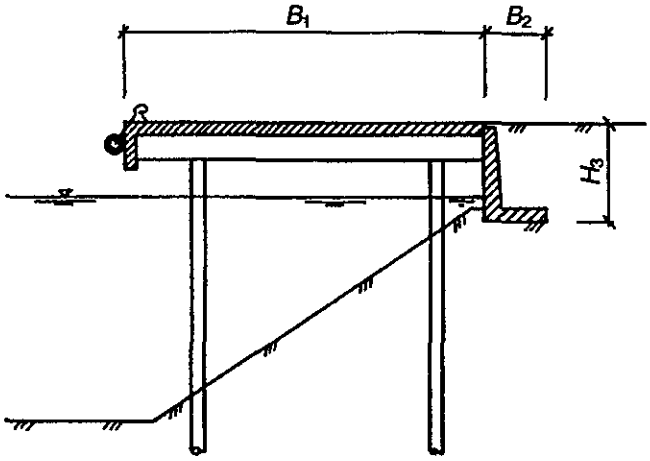


Fig. 11.2. Precast anchoring structure on levelled underwater base

Generally, the economically best solution for an open berth structure will be obtained when the total width *B* is as small as possible in relation to the height of the berth front. The different characteristic dimensions influencing the structural dimension and design are discussed below.

H is the height of the berth front and is determined by the necessary water depth and height of the berth surface above lowest astronomical tide LAT.

H_1 is the depth between LAT and the bottom of the harbour basin, is determined by the draft of calling ships when fully loaded plus an over-depth to cover the trim of the ship, wave height and safety margin against sea bottom irregularities.

H_2 is the elevation of the top of berth platform above LAT and is determined by the elevation of the area behind the berth and or by types of ship, which will call at the berth structure. Top of platform should not be at a lower level than the highest observed water level plus 0.5 m.

H_3, this distance is determined by the location of the rear wall or the retaining slab above LAT. The bottom of the slab should not lie lower than mean sea level. The rear structure should be placed at such a level that concreting can be carried out on dry land. In case it is necessary for the rear structure to lie lower, the construction can be carried out with pre-fabricated elements placed on a levelled base under water, as shown in Fig. 11.2. By increasing the height *H* and the angle of the slope, the total width *B* will be reduced, but the economic benefit by reducing *B* can easily be eliminated by higher installation costs because of underwater work.

a is the slope should start about 1.0 m behind the berth front so that the toe of the slope is kept more away from the turbulence caused by propellers. Possible rocks falling off the slope will then probably be prevented from falling outside the berth front line.

b is the distance *b* is determined by the steepness of the slope. The angle *a* will normally vary between 38.7° (1:1.25) and 29.7° (1:1.75), depending on the materials of which the slope is made, whether covered with blasted rock, etc. Usually the angle is 33.7° (1:1.5). The stability of the slope itself, the coarseness of the materials and the danger of erosion from waves and propeller turbulence determine the angle of the slope.

c is a very exposed point because of the danger of slides in front of the rear wall or retaining friction slab. The width *c* of the shoulder must be at least 3.0 m, and the shoulder itself must be well covered. The width *c* must also be sufficient to ensure the stability in front of the retaining slab.

d is the width of the anchor or friction slab, which shall resist the horizontal load, depends on the frictional angle in the ground, the safety factor wanted against sliding and the vertical load acting on the retaining slab.

e is the distance between the berth line and the centreline of the first row of supporting columns or piles is determined by the possibility that a ship with a long bulbous bow protruding well forward of the forepeak below the water line and with a markedly flared hull at the bow, like larger container ships, should hit the columns or piles if berthing with a large berthing angle. To avoid this possibility the centreline of the columns or piles should lie at least 2 m behind the berth line.

B_1 is the width of the berth platform is determined by the other main characteristics of the berth structure.

B_2, this dimension is determined by the thickness of the rear wall or the slope of the relief plate and by the width of the anchor slab. The angle β should not be larger than 15° (1:3.75) because of the magnitude of the vertical forces one gets when the horizontal load is transmitted by way of the sloping or settlement slab to the anchor slab. This settlement slab should have an upper horizontal part lying about 30 cm under top of the filled apron.

Even if it is generally desirable that the width *B* be reduced as much as possible, it should not be forgotten that the horizontal forces must be transmitted to the anchor slab in a satisfactory way. It is necessary to design the transitions between the berth platform and the settlement slab and between the settlement slab and the anchor slab in such a way that the settlements of the fill under the anchor slab are taken care of. As a rough guideline the settlement of a filling consisting of

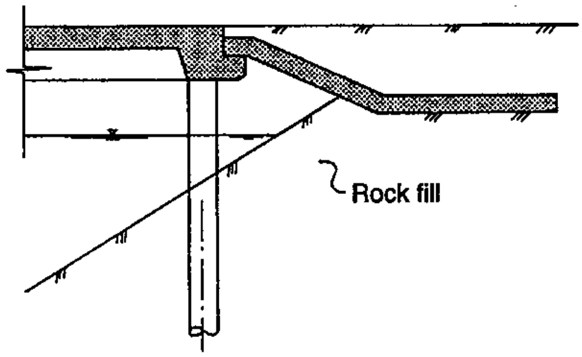


Fig. 11.3. Possible precast anchoring structure with minimal ground settlements

stone-sand friction materials should be 1–3 per cent of the height of the filling. Filling made of rock and stones will have a volume ratio between filling placed with good compaction to solid rock of about 1.3–1.5, and between filling placed without any compaction underwater to solid rock of about 1.5 to 1.7.

If the settlements are expected to be minimal, the anchoring structure can be shaped as shown in Fig. 11.3. Where greater settlements are expected the anchoring structure must be shaped as shown in Fig. 11.4.

If the anchoring structure is shaped as shown in Fig. 11.5, where a part of the rear wall is founded on piles, the anchor slab must be shaped in such a way that the forces can be transferred through friction between slab and fill.

If the anchoring structure is shaped as shown in Fig. 11.6, the horizontal forces are taken up by batter piles, while the anchor slab only makes possible the transfer of adequate stabilizing weight to the batter piles.

In cases where the width B_2 of the anchoring structure is made very short, it can be shaped as shown in Fig. 11.7. The rear wall and the berth platform are cast in one, and batter piles under the rear wall

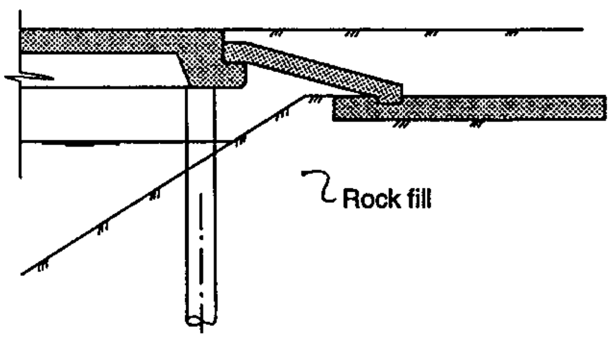


Fig. 11.4. Anchoring structure where considerable ground settlements are expected

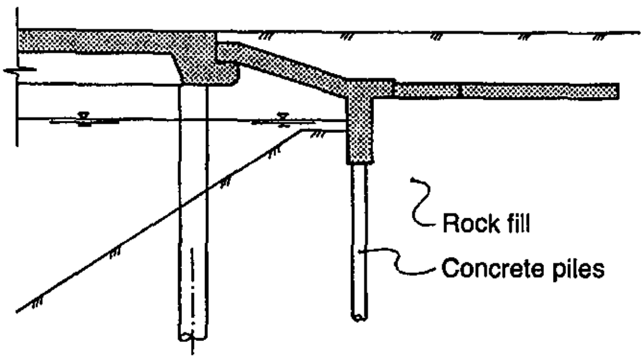


Fig. 11.5. Anchoring structure partly on piles

take up the horizontal forces. This design is labour intensive and costly. It should also be noted that a vertical wall would attract forces onto the structure from the fill behind.

B is the total width of the berth structure, B, depends therefore on several variable factors. The designer should try to choose a berth design, which will give a clearly set-out load bearing and the easiest possible construction work for the contractor. Sophisticated solutions should be avoided. In piers and jetties the total width B is equal to the berth platform width B_1 and the horizontal loads must be taken up by lamellas or by batter piles under the berth. The deciding factor for determining the width B will therefore be the port activities that will be carried out on the berth structure. Where the pier is so wide that one can use fill for the middle part, or where the quay lies parallel to the shore, the total width B will be determined by the dimensions shown in Fig. 11.1.

Foundations for column, pile and lamella berths, respectively, will be described in the following sections. The berth platforms are similar for the three types of structure and will be described in a separate section.

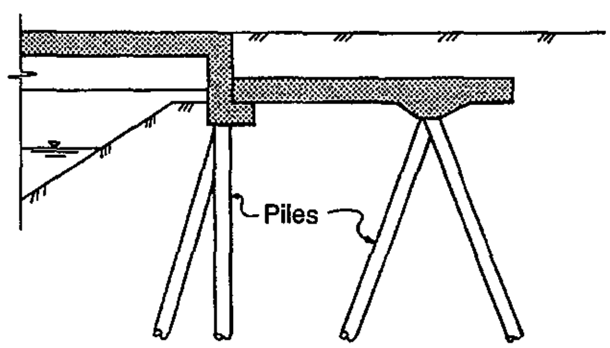


Fig. 11.6. Use of batter piles

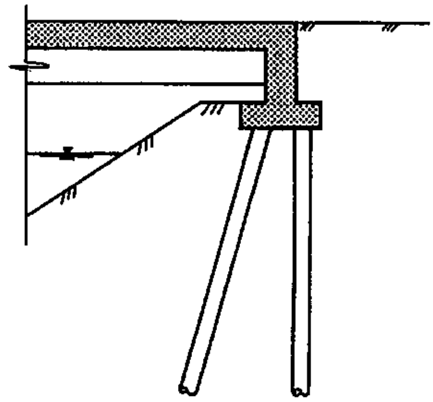


Fig. 11.7. Very short structure anchoring

11.2 Column berths

The column berths have the berth platform founded on columns cast in situ.

The column berth is a type of berth which is special to Norway where contractors have specialized in the construction of long and slender columns cast in situ underwater. In other countries this type of construction is met with scepticism and the majority of berths are built as solid berths or as pile berths. This fact is confirmed whenever one reads foreign literature on open berth structures where very little, if anything, is said about slender concrete columns cast underwater using the tremie pipe method. Figure 11.8 shows a cross-section through a large open berth anchored in the rockfill by an anchor slab.

In recent years, the ratio between column berths and pile berths has changed because, nowadays, open berth structures on tubular steel piles are preferred if the ground conditions permit. This is not because of scepticism

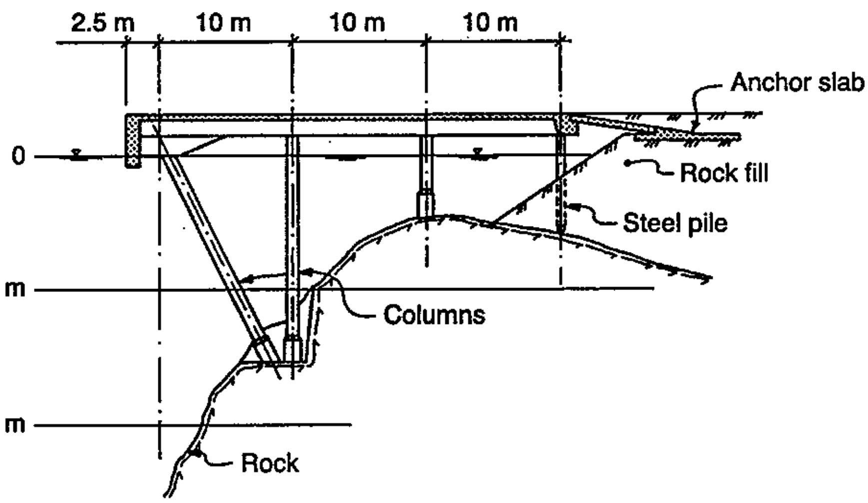


Fig. 11.8. Column berth

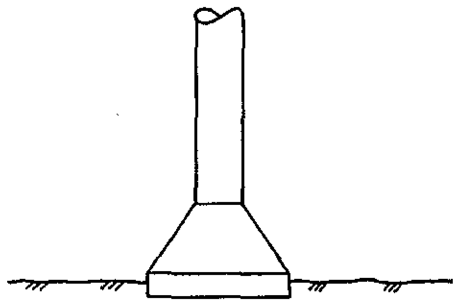


Fig. 11.9. Foundation on hard moraine or rock

regarding the durability and performance of concrete column berths, but because structures on tubular steel piles are economically more advantageous, especially as expensive underwater work is avoided.

In column berths the berth platform is supported on in situ cast columns which are either founded directly on moraine or rock, or which penetrate through loose deposits in vertical wells down to rock. Alternatively, each column is supported on a group of friction or point-bearing piles. The various methods are described below.

Moraine

As shown in Fig. 11.9, the column with a widened foot is placed directly on the moraine. To make certain that the moraine has sufficient load-bearing capacity it is recommended that, in addition to removing possible loose deposits, the upper layer of the moraine should be removed. However, this must be done carefully so as not to disturb the underlying material and cause settlement of the column. The size of the foundation will depend on the allowable compressive stress under the foundation and on considerations related to the casting technique.

As a rough guideline the following allowable pressures can be assumed for a rough estimate of the size of a foundation:

Rock	5000–30 000 kN/m^2
Hard moraine above groundwater level	500–800 kN/m^2
Hard moraine below groundwater level	400–600 kN/m^2
Compressed rock/stone filling above groundwater level	300–500 kN/m^2
Compressed rock/stone filling below groundwater level	200–300 kN/m^2
Dense sand and gravel above groundwater level	300–500 kN/m^2

Dense sand and gravel below groundwater level	200–300 kN/m^2
Loose to medium dense sand and gravel above groundwater level	150–250 kN/m^2
Loose to medium dense sand and gravel below groundwater level	100–150 kN/m^2
Hard clay (dry crust)	150–250 kN/m^2
Medium-soft clay	80–150 kN/m^2
Soft clay	20–80 kN/m^2

Rock

Where rock is uncovered, or where the seabed layer above rock is such that by dredging rock becomes easily visible, the column is founded directly on rock. Whether the rock surface is horizontal or sloping, one should always blast a shelf or a niche as shown in Fig. 11.10. This is especially important if the column is to be placed in a backfill slope. The column foot should be shaped according to the code of practice for load bearing underwater concrete constructions, as described in Chapter 17. Before casting, all loose deposits and mud should be removed. Anchoring bolts between column and rock are recommended. However, the foundation should be designed in such a way that the load pressure is borne by the concrete alone and not by the anchoring bolts. To achieve an adequate stability of the columns during the construction phase, i.e. prior to beams and slab being cast, it is advisable to place an anchor bolt at each corner of the column to avoid tilting.

Wells

The decisive factor as regards the method to be used, i.e. whether one should shaft a well or dredge, depends on the dredging characteristics of

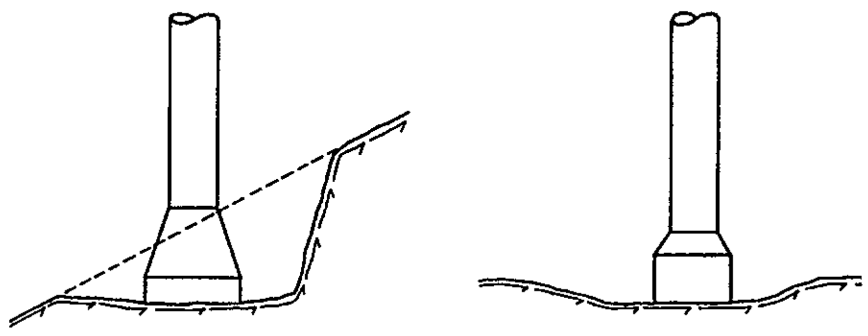


Fig. 11.10. Foundation on rock

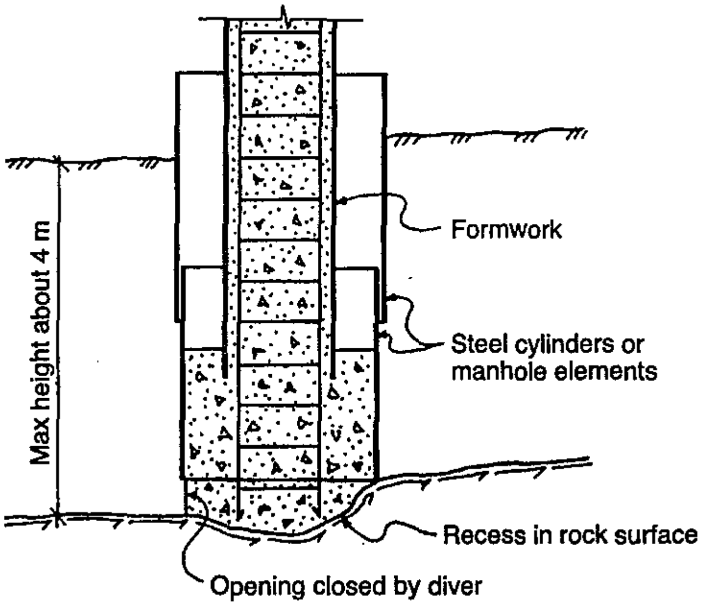


Fig. 11.11. Well foundation

the soil and the contractor's equipment. General practice indicates that if the thickness of the soil is between about 1.5–4.0 m, it could be worthwhile to use vertical wells as shown in Fig. 11.11. The dredging inside the well can be done either by grabbing or by a large suction pump. As soon as the dredging work is finished, a recess must be either blasted or chiselled into the rock surface and the rock cleared. The widening of the column at the bottom is achieved by stopping the formwork about 0.5–0.7 m above the rock. The well is formed by the use of steel cylinders on manhole elements of diameter at least 1.5 m allowing the diver to operate inside them.

Piles

If the depth from seabed to rock or load resistant stratum is more than about 4.0–5.0 m, it will probably pay to use piles as foundation for the columns. The simplest and often most economical method is to use untreated wooden piles as shown in Fig. 11.12. In order to prevent wood-borer attacks in the wooden piles, the foundation should be dredged at least 0.5 m down into the seabed. In order to avoid a construction joint between the column foot and the column they must be cast in one and the same operation.

As regards the precision in the placing of the columns, the centre of the column should be placed within a tolerance of only 5 cm from the theoretical column centre.

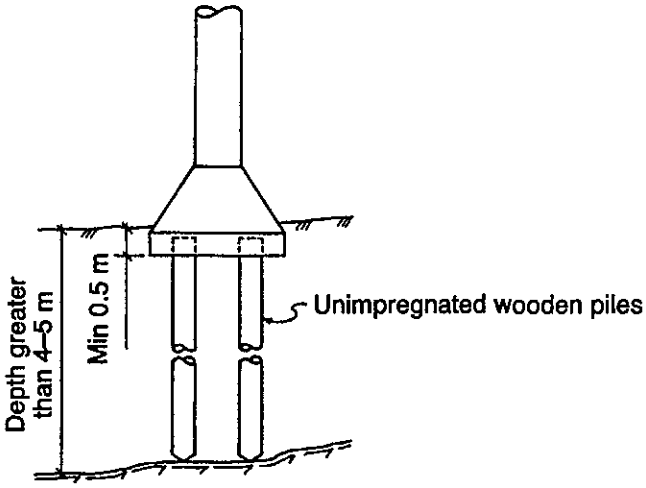


Fig. 11.12. Pile foundation

Formwork

Berth columns should be shaped in such a way that a multi-use formwork system can be applied, requiring little diving work for installation underwater. The cross-sections of the columns can be both rectangular and circular, but with the advanced formwork systems which are in use nowadays, the circular cross-section is probably the simplest and most economical. A circular cross-section will, when using the same amount of concrete as a square cross-section, have about 13 per cent less formwork surface and therefore about 13 per cent less surface exposed to frost and chemicals than a square cross-section. A well-designed formwork system with a circular cross-section is simple to erect and dismantle, which is very important particularly when divers have to be used. Furthermore, circular stirrups are easy to bend and install inside the form.

Usually the formwork for columns, with the reinforcement installed inside, is made ready ashore as shown in Fig. 11.13 and then towed to its place in the structure. Alternatively it can be lifted into place by crane as shown in Fig. 11.14. It is preferable to standardize the column cross-sections and vary the amount of reinforcement in the columns in relation to the column length and load. When cast underwater, the column should not have a diameter of less than 70 cm. The cross-section, the reinforcement and the shaping of the columns should be as recommended in Chapter 17.

All formwork placed under the tidal zone should be removed, so that the concrete casting can be closely examined by divers. In the tidal zone the formwork should remain for the protection of the concrete. Experience has shown that it is only in this zone that the concrete is severely exposed to frost destruction etc., as shown in Fig. 11.15.

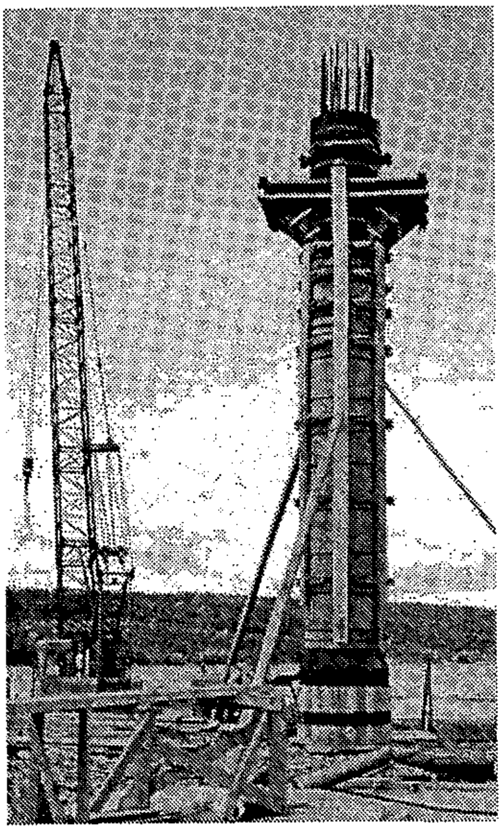


Fig. 11.13. Column formwork with reinforcement

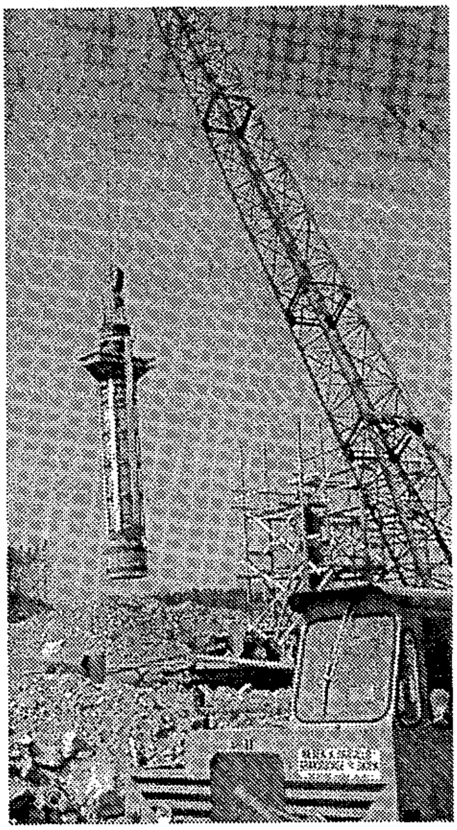


Fig. 11.14. Column formwork with reinforcement being lifted into place

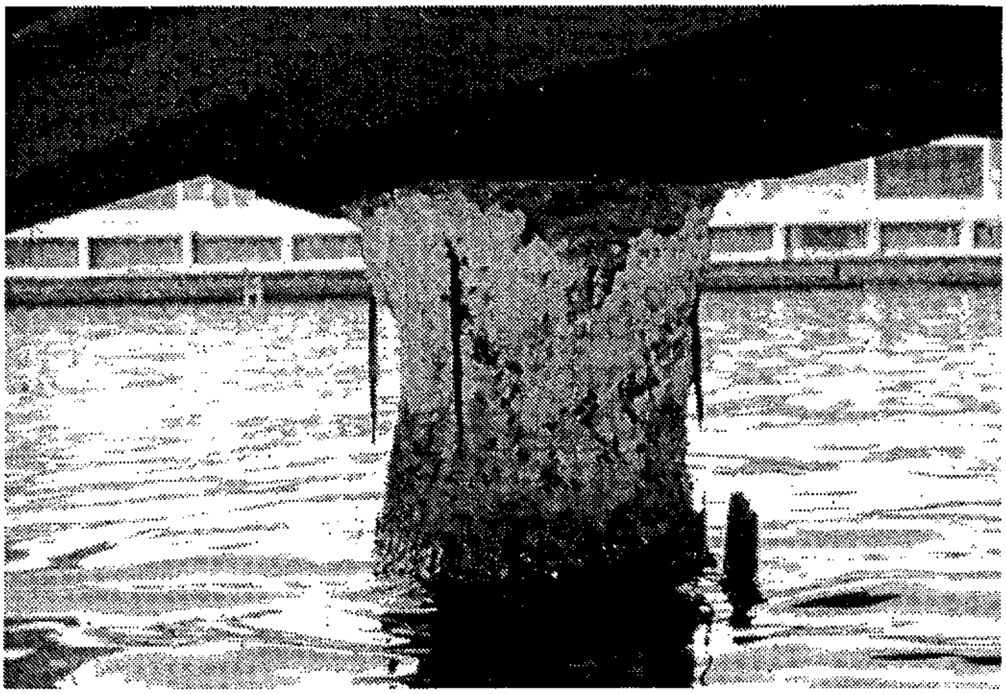


Fig. 11.15. Column with hour-glass shape

Permanent formwork in the tidal zone should be made of material that is fully impregnated and held in place by copper ties and copper pegs. Instead of fully impregnated wooden material, plastic pipes and thin steel tubes have been used in the tidal zone with good results. Such permanent formwork covers the area from the bottom of the berth platform beams down to 50 cm under lowest astronomical tide LAT, as shown in Fig. 11.16.

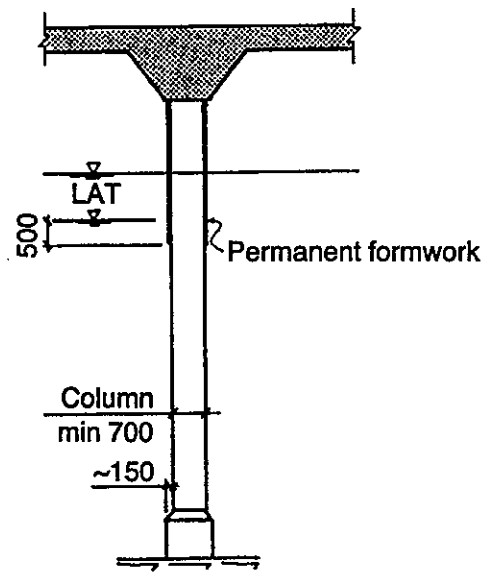


Fig. 11.16. Column foundation

11.3 Pile berths

In smaller open berth structures where the water depth is small and the load bearing capacity of the berth platform is limited, concrete or timber piles usually support the platform. If the platform is required to withstand greater live loads, the piles must be placed so closely together that other structural solutions are more economical. The usual alternative has been concrete columns of a minimum 70 cm diameter founded as described above for column berths.

Where the thickness of seabed material is more than about 3–4 m, allowing a foundation directly on piles, piles of much greater cross-sectional dimensions could be used. The piles that reach up to the berth platform, as shown in Fig. 11.17, are either concrete piles or tubular steel piles filled with concrete. Generally, concrete piles are the more expensive alternative per lin m of berth, due to a higher transportation cost if the pile factory is not located near the site, and the fact that the concrete piles must be strengthened and/or protected in the tidal zone. The construction of a pile berth with tubular steel piles is shown in Fig. 11.18.

Tubular steel pipes can be driven into harder and more difficult strata than concrete piles. For instance, a 70 cm diameter tubular steel pile can penetrate a 20 m thick rubble fill consisting of stones with diameter as large as 50 cm. Piles of diameter 50–80 cm are the most commonly used.

Table 11.1 shows spirally welded steel pipe piles with outside diameters from 60.3–1219.0 mm. Prefabricated pile units complete

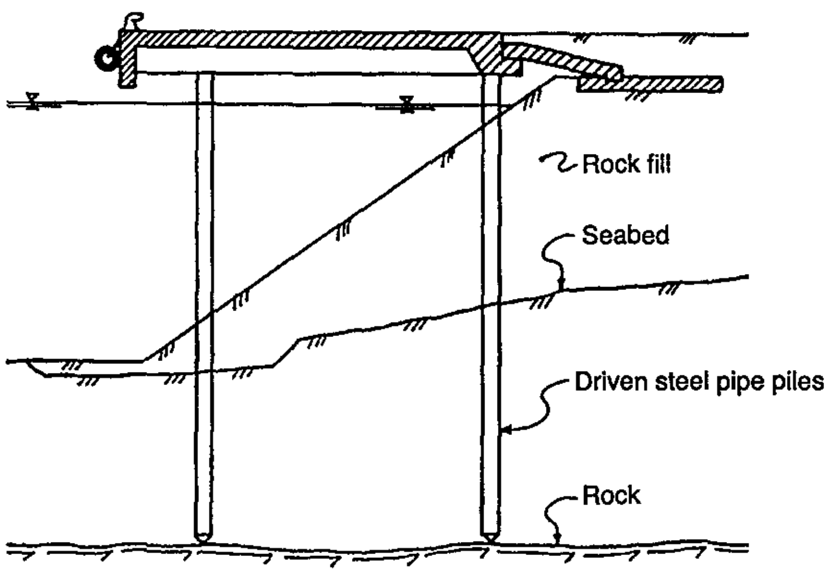


Fig. 11.17. Pile berth

Table 11.1. Steel pipe piles. Dimensions and cross-section values. Courtesy of Rautaruukki, Finland

D (mm)	t (mm)	M (kg/m)	A (cm^2)	A_u (m^2/m)	I (cm^4)	W (cm^3)	W_p (cm^3)	i (cm)	I_v (cm^4)	W_v (cm^3)
406.4	8.0	78.6	100.13	1.277	19 873.89	978.05	1269.95	14.09	39 747.79	956.09
406.4	10.0	97.8	124.53	1.277	24 475.81	1204.52	1571.66	14.02	48 951.63	2409.04
406.4	12.5	121	154.68	1.277	30 030.67	1477.89	1940.12	13.93	60 061.33	2955.77
457.0	8.0	88.6	112.85	1.436	28 446.36	1244.92	1612.98	15.88	56 892.73	2489.83
457.0	10.0	110	140.43	1.436	35 091.32	1535.73	1998.42	15.81	70 182.65	3071.45
457.0	12.5	137	174.55	1.436	43 144.80	1888.18	2470.40	15.72	86 289.61	3776.35
508.0	8.0	98.7	125.66	1.596	39 279.96	1546.46	2000.17	17.68	78 559.92	3092.91
508.0	10.0	123	156.45	1.596	48 520.25	1910.25	2480.37	17.61	97 040.49	3820.49
508.0	12.5	153	194.58	1.596	59 755.40	2352.57	3069.65	17.52	119 510.80	4705.15
508.0	14.2	173	220.29	1.596	67 198.62	2645.62	3463.46	17.47	134 397.25	5291.23
559.0	8.0	109	138.48	1.756	52 564.94	1880.68	2428.98	19.48	105 129.89	3761.36
559.0	10.0	135	172.47	1.756	65 001.14	2325.62	3014.34	19.41	130 002.28	4651.24
559.0	12.5	168	214.61	1.756	80 161.82	2868.04	3733.93	19.33	160 323.63	5736.09
559.0	14.2	191	243.04	1.756	90 230.71	3228.29	4215.61	19.27	180 461.42	6456.58
610.0	8.0	119	151.30	1.916	68 551.35	2247.59	2899.40	21.29	137 102.71	4495.17
610.0	10.0	148	188.50	1.916	84 846.56	2781.85	3600.33	21.22	169 693.13	5563.71
610.0	12.5	184	234.64	1.916	104 754.73	3434.58	4463.23	21.13	209 509.47	6869.16
610.0	14.2	209	265.79	1.916	118 003.90	3868.98	5041.64	21.07	236 007.79	7737.96
660.0	8.0	129	163.87	2.073	87 087.94	2639.03	3401.00	23.05	174 175.89	5278.06
660.0	10.0	160	204.20	2.073	107 870.51	3268.80	4225.33	22.98	215 741.02	6537.61
660.0	12.5	200	254.27	2.073	133 306.41	4039.59	5241.35	22.90	266 612.83	8079.18
660.0	14.2	226	288.10	2.073	150 263.08	4553.43	5923.17	22.84	300 526.15	9106.85
711.0	8.0	139	176.68	2.234	109 162.15	3070.67	3953.84	24.86	218 324.30	6141.33
711.0	10.0	173	220.23	2.234	135 301.41	3805.95	4914.34	24.79	270 602.81	7611.89

711.0	12.5	215	274.30	2.234	167 343.25	4707.26	6099.43	24.70	334 686.49	9414.53
711.0	14.2	244	310.85	2.234	188 735.23	5309.01	6895.48	24.64	377 470.47	10 618.02
762.0	8.0	149	189.50	2.394	134 683.01	3534.99	4548.30	26.66	269 366.01	7069.97
762.0	10.0	185	236.25	2.394	167 028.35	4383.95	5655.37	26.59	334 056.71	8767.89
762.0	12.5	231	294.33	2.394	206 730.99	5426.01	7022.53	26.50	413 461.99	10 852.02
762.0	14.2	262	333.60	2.394	233 271.23	6122.60	7941.66	26.44	466 542.46	12 245.21
762.0	16.0	294	374.98	2.394	260 973.25	6849.69	8905.62	26.38	521 946.50	13 699.38
813.0	8.0	159	202.32	2.554	163 900.55	4031.99	5184.37	28.46	327 801.09	8063.99
813.0	10.0	198	252.27	2.554	203 363.90	5002.80	6448.42	28.39	406 727.81	10 005.60
813.0	12.5	247	314.36	2.554	251 860.34	6195.83	8010.65	28.31	503 720.69	12 391.65
813.0	14.2	280	356.35	2.554	284 314.90	6994.22	9061.71	28.25	568 629.80	13 988.43
813.0	16.0	314	400.62	2.554	318 221.72	7828.33	10 164.71	28.18	636 443.45	15 656.67
914.0	8.0	179	227.70	2.871	233 651.32	5112.72	6566.86	32.03	467 302.64	10 225.44
914.0	10.0	223	284.00	2.871	290 147.16	6348.95	8172.49	31.96	580 294.31	12 697.91
914.0	12.5	278	354.02	2.871	359 708.40	7871.08	10 159.43	31.88	719 416.80	15 742.16
914.0	14.2	315	401.41	2.871	406 344.46	8891.56	11 497.84	31.82	812 688.92	17 783.13
914.0	16.0	354	451.38	2.871	455 141.80	9959.34	12 903.83	31.75	910 283.61	19 918.68
1016.0	10.0	248	316.04	3.192	399 849.67	7871.06	10 120.69	35.57	799 699.33	15 742.11
1016.0	12.5	309	394.07	3.192	496 123.06	9766.20	12 588.30	35.48	992 246.11	19 532.40
1016.0	14.2	351	446.91	3.192	560 761.98	11 038.62	14 252.12	35.42	1 121 523.96	22 077.24
1016.0	16.0	395	502.65	3.192	628 479.38	12 371.64	16 001.37	35.36	1 256 958.76	24 743.28
1219.0	10.0	298	379.82	3.833	694 014.28	11 386.62	14 617.14	42.75	1 388 028.57	22 773.23
1219.0	12.5	372	473.79	3.833	862 180.91	14 145.71	18 196.18	42.66	1 724 361.82	28 291.42
1219.0	14.2	422	537.47	3.833	975 333.95	16 002.20	20 612.87	42.60	1 950 667.90	32 004.40
1219.0	16.0	475	604.69	3.833	1 094 091.08	17 950.63	23 156.71	42.54	2 188 182.16	35 901.27

M = weight, A = cross-section area, A_u = external area, I = moment of inertia, W = section modulus, W_p = elastic modulus, i = radius of gyration, I_v = torsion modulus, W_v = section modulus in torsion and theoretical density = 7.85 kg/dm^3. The cross-sectional rates have been calculated using nominal dimensions D and t.

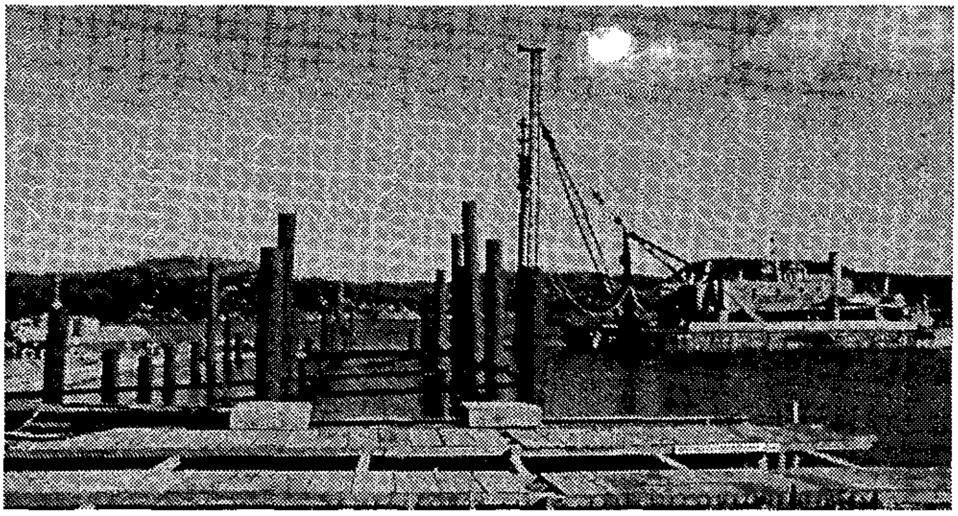


Fig. 11.18. Construction of a pile berth

with fittings are available in lengths up to 26 m for delivery direct to the construction site. The steel piles are made of general structural steel and high-strength gas pipeline steel in accordance with standard EN 10219, as shown in Table 11.2. The steel grade most commonly used for pipe pile is S355J2H. Steel pipe piles of dimensions between ∅600 and ∅800 are usually used. The pipes should have a minimum wall thickness of 10 mm and should be delivered to the site in lengths of minimum 12 m. The steel material should be normalized steel and the quality of the steel should be S355J2H. A certificate from the manufacturer is required. Due to the fact that the pipes could be joint welded at the site, the tolerances should be:

(a) Diameter is ± 2 mm.
(b) Ovality ($d_{max} - d_{min}$) is 5 mm.
(c) Deviation from 90° end cuts is 2.5 mm.
(d) Linearity is 5 mm over 5 m length.

The steel piles can withstand heavy pile driving and can easily be joined by butt-welding while in the rammer, as shown in Fig. 11.19. The steel thickness depends on the driving conditions and how much driving energy is required, but usually the 50–80 cm diameter piles have steel thickness of 8–12 mm. Pile driving requires heavy equipment. During driving the tubular steel piles are filled with water. The steel pile points are either of a conic or a flat type. The flat type is shown in Fig. 11.20.

The pile sections can be spliced in the rammer as shown in Fig. 11.19. If the lower pile has been driven into hard soil, the upper 100 mm of the pipe should be cut off and the contact surface should be worked to even, right-angled planes before the upper pile is welded to the lower pile.

The welding should be carried out by the use of basic electrodes corresponding to, for instance, OK 48.30. The electrodes must be stored in warm containers to prevent moisture. In order to secure the relative positions of the pipe ends an inside pipe of 3 mm thickness and 60 mm length should be provided as shown in Fig. 11.21.

In order to facilitate the control of the linearity, the splice should be carried out at least 1.5 m above the water level. Maximum allowable angular deviation after splicing should be 1:250, measured over a length of 3.0 m. This requirement is valid for the entire length of the pile. All splices should be prepared for V-welds.

The piles will be equipped with **pile shoes** according to the specified bearing capacity and the ground conditions. Open-ended piles will usually be equipped with either an external or an inside reinforcement ring. Bottom plates are often used where the piles are mainly end bearing in a boulder-free soil layer. When the piles are to be driven through rocky moraine or into bedrock, rock shoes fitted with a structural steel dowel are used to prevent damage to the pile end and to centre the pile load. The rock shoes with the hardened steel dowel are used especially to prevent the pile from skidding on the sloping rock surface.

Figures 11.20 and 11.21 show the pile point and the construction details of the point. Figure 11.22 shows different pile shoes from Rautaruukki.

Before the welding, the steel parts shall be preheated to 150 °C and a slow cooling after welding must be secured. The hard facing of the pile shoe should be as shown in Fig. 11.21 and should be made in the following steps:

(a) 2–3 welding layers on the base material with hard weld electrode giving 200–250 HB
(b) 2 layers giving 300–350 HB
(c) 2 layers giving 400–450 HB.

All welds should be controlled by ultrasound with respect to hydrogen cracks. Tests by the use of magnetic powder or penetration liquid are recommended for detecting surface cracks. The hard facing of the shoe should be controlled by the use of magnetic powder (with coil) or by the use of penetration liquid. The control of the hardness on test specimens can be accepted provided that the same welding procedure has been applied for both the hard facing and the specimen.

Table 11.2. Steel pipe piles, steel grades

Steel grade	Standard	Chemical composition, maximum				Mechanical properties			Imp. strength	
		C (%)	Mn (%)	P (%)	S (%)	R_{eH} Min. (N/mm^2)	R_m (N/mm^2)	A_5 Trav. min. (%)	T°C	KV (J min.)
S355J2H	EN 10219	0.22	1.60	0.035	0.035	355	490–630	20	−20	27
X60	API 5L	0.15	1.60	0.03	0.03	413	≥517	18	−20	27
X70	API 5L	0.15	1.70	0.03	0.03	482	≥565	18	−20	27

The **precision,** with which the tubular steel **piles are placed,** will depend on the characteristics of the fill material and the contractor's experience. However, the centre of the pile should be placed within a tolerance of 30 cm limit of the theoretical centre and the deviation from the vertical shall not exceed 2° or approximately 30:1. The

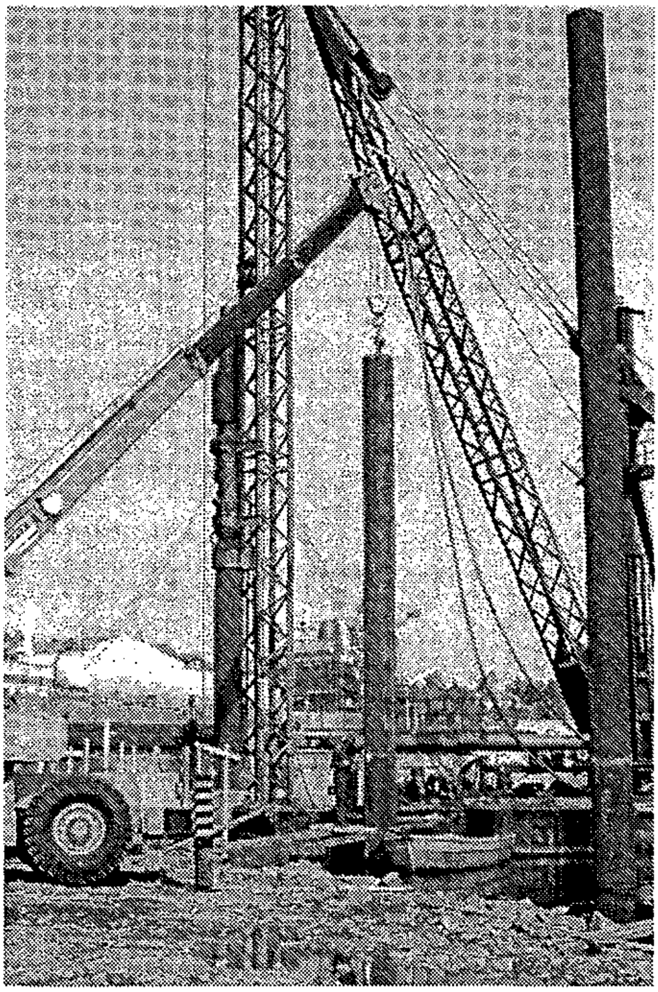


Fig. 11.19. Welding of steel piles

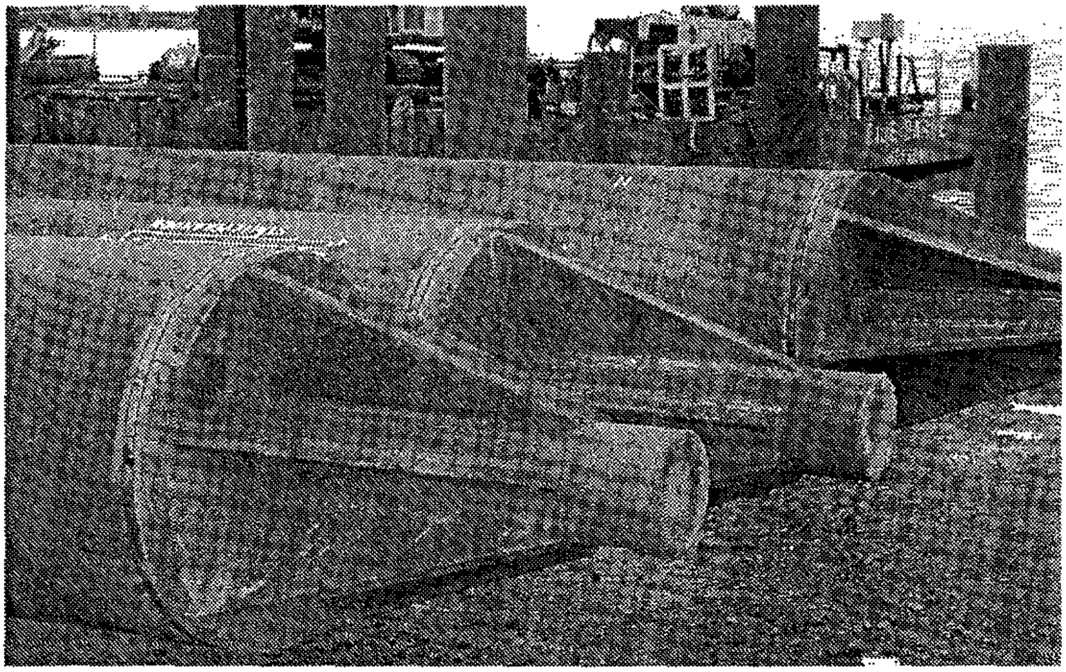


Fig. 11.20. Steel pile point

bottom side of the berth platform beams must have sufficient width to accommodate the tolerance of the piles. If the contractor can provide a flexible formwork system, there will be no difficulty in accommodating the pile's position.

The **piles** should usually be **driven** by ordinary gravity hammers, or single-acting pneumatic hammers with a weight of 40 kN or more. The equipment must be adjustable to ensure that the driving force acts along the axis of the pile.

A pile cap containing hard wood should, during the pile driving, protect the pile top. In the case of large deformations of the pipe top during the hammering, this must be cut off and ground to an even plane before the driving continues. Because the submerged weight of an empty pile pipe is less than unity, the piles must be filled with water during driving.

If the piles are embedded in a thin soil layer above the bedrock, this layer may not give a sufficient lateral support of the piles, or sufficient anchoring against uplift when the piles are emptied before concreting. The contractor should therefore take the necessary measures to secure the positions of the piles in all directions.

A general preliminary pile driving procedure may be as follows:

During the driving through, for example, fill and moraine, the drop height should not exceed 600 mm. When the pile tip reaches the rock

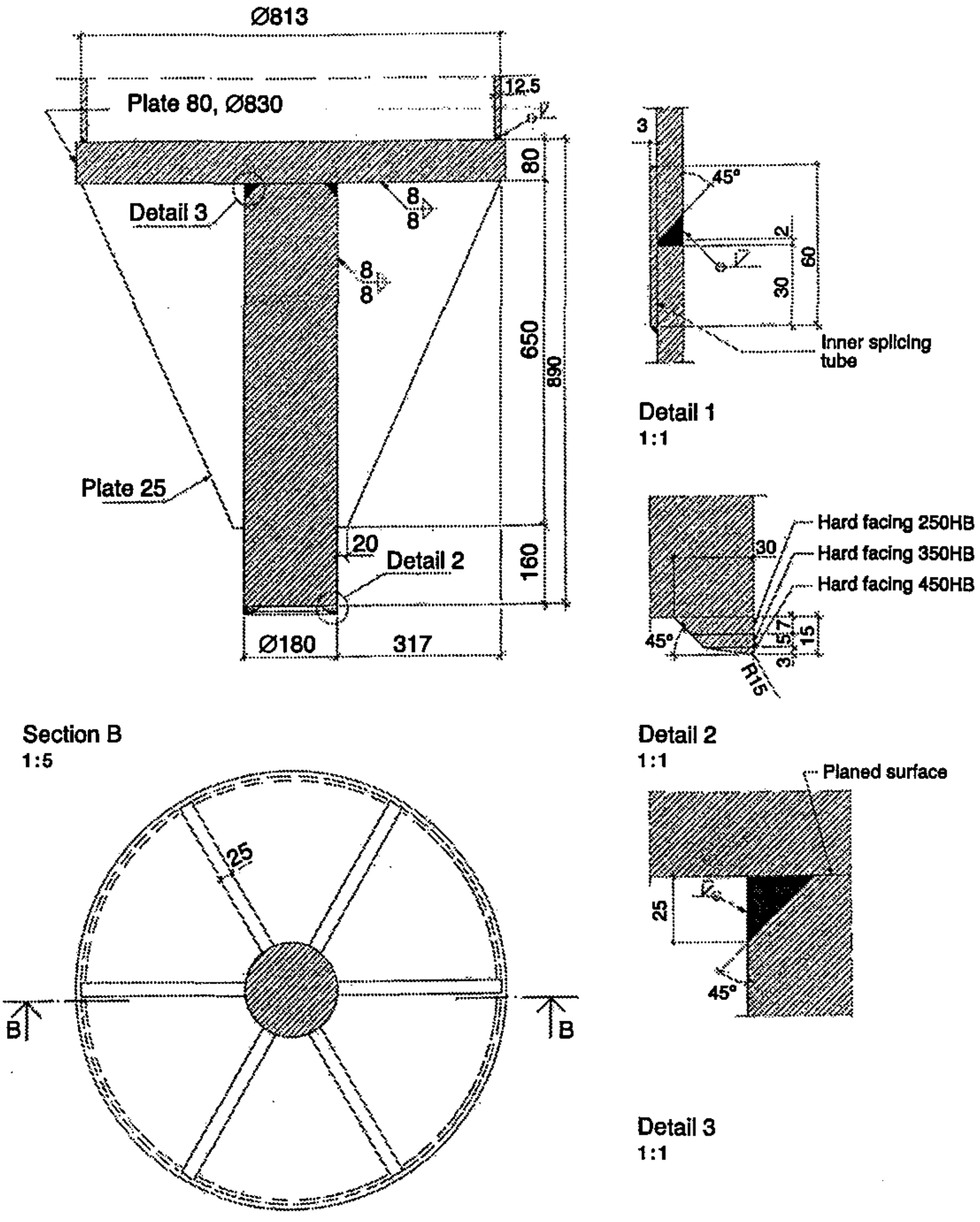
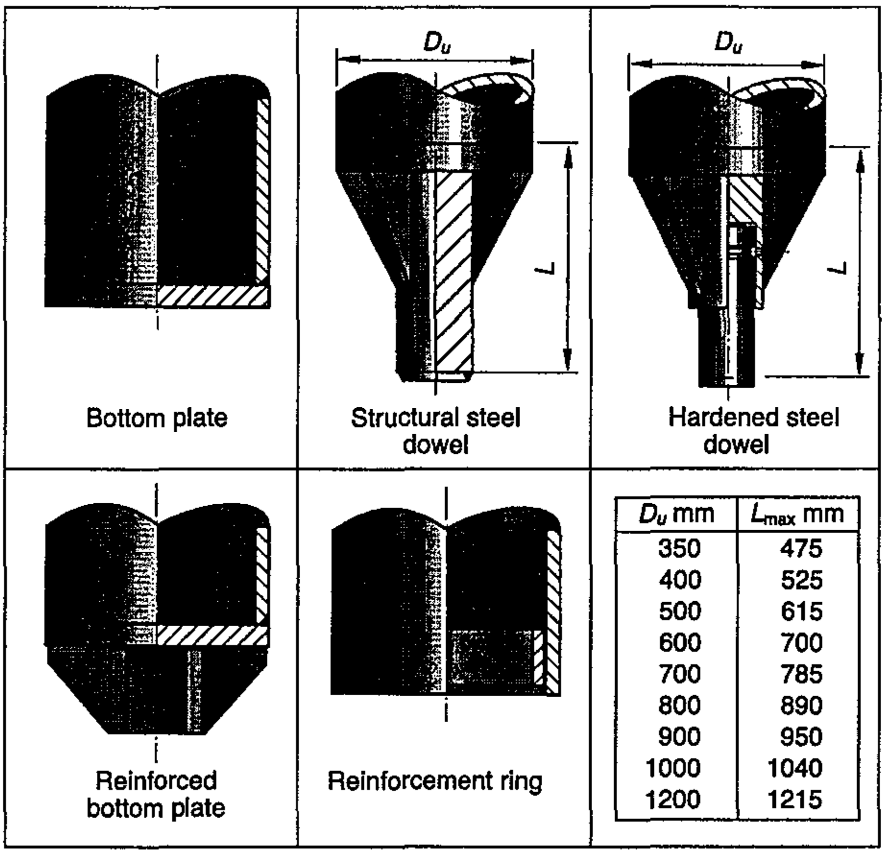


Fig. 11.21. Detail of the pile point

surface the height should be reduced to 300 mm. The penetration/series with 10 blows should be measured, and when the penetration is 5 mm/series and has a constant or decreasing tendency, the drop height should be raised to 500 mm. When 5 mm/series are obtained again, the pile shoe rock contact is regarded as satisfactory. In case of increasing penetration it should be returned to a 300 mm drop-height series and the whole procedure repeated. In order to ascertain that the last blows have not caused any failure in the rock or pile, 3 series with drop



D_u mm	L_{max} mm
350	475
400	525
500	615
600	700
700	785
800	890
900	950
1000	1040
1200	1215

Fig. 11.22. Different pile shoes from Rautaruukki, Finland

height 300 mm should finish the driving. The penetration for each series should not exceed 3 mm.

The driving procedure and the criteria given may be changed during the performance depending on the experiences made at the site, and the contractor's equipment. The contractor for each pile should prepare a complete piling record.

The control by the contractor of the piling work should take care of both the quality of the piles and pile shoes, the handling and the driving operation. The person responsible for the control and the filling in of the piling records must be an experienced controller.

Emptying or lowering the water level inside the pile pipe should control the water tightness of the piles. If there is leakage, too great curvature, or other unfavourable conditions, the Client shall decide if it is necessary to replace the pile with a new one. Replaced piles should be satisfactorily located and be included in the pile record.

When the driving of a pile is finished, the top of the pile should be levelled. Before concreting, the piles should be relevelled in order to confirm that the pile tips are still in contact with the rock. If the upheaval is more that 3 mm, this should require new redriving of the

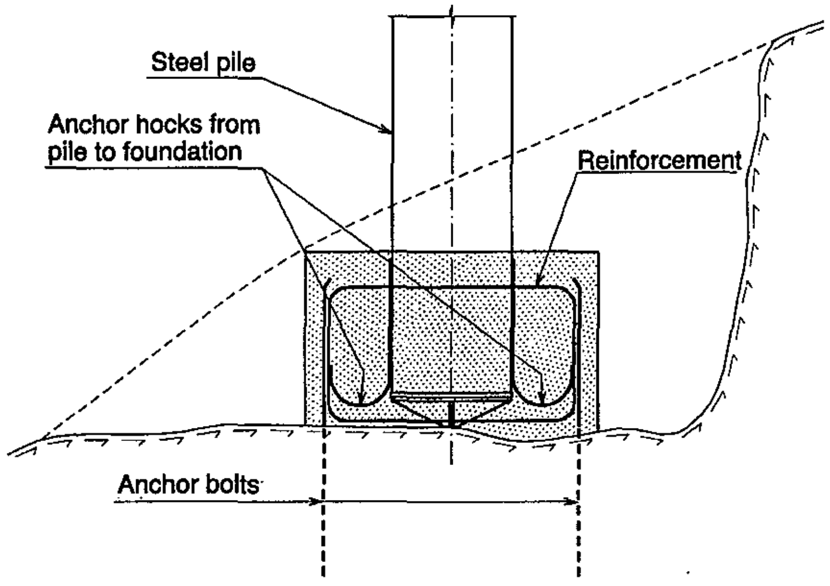


Fig. 11.23. Pile in concrete foundation

pile. The redriving shall comply with the requirements laid down in the preliminary driving procedures.

The tolerances on the cut-off levels of the top of the pile should be ±5 mm. No final cut-off or concreting of the piles shall be performed until the pile has been inspected and found acceptable.

Some particular problems can arise with concrete-filled steel pipe piles during driving of the steel piles. The problems are due to the fact that the cross-sectional area of the steel pipe is too small to bring down a dynamic load of the same magnitude as the design load of the concrete-filled pipe. The maximum load to be transmitted down is the yielding stress of the steel multiplied by the cross-sectional area. The thickness of the pipe wall must, therefore, in many cases be increased in order to meet with the requirements mentioned above.

If some of the seabed under the berth is bare rock, the piles can be put in place both with and without points or shoes to a foundation, fixed and filled with concrete as is done in underwater concreting. The piles are fixed by bolting to the rock in a concrete foundation, sealed at the foot, emptied of water, and then filled with concrete, as in a dry formwork, as shown in Fig. 11.23.

The term load-bearing **capacity of piles** normally means the ability of the ground to bear the pile load contrary to the structural capacity of the pile. The load-bearing capacity can be evaluated or calculated in different ways. For piles in soil, i.e. friction piles, there are many static formulas available. Most of them divide the capacity in shaft friction and point resistance. These again are based on soil parameters derived from laboratory tests, and/or soundings, for instance Standard Penetration Tests or Cone Penetration Tests. In this way, the ultimate capacity and the necessary pile lengths can be calculated. The design load capacity is obtained by dividing the ultimate capacity by a material factor of 1.6–2.0 depending on the type and quality of soil data.

The typical formula for the ultimate bearing capacity is:

$$Q_u = Q_s + Q_p$$

where Q_s is the ultimate shaft friction and Q_p is the ultimate point resistance.

$$Q_s = \beta \times p' \times A_s$$

where

β = shaft friction coefficient. For rough estimates the value of this vary typically between 0.30 and 0.15, the lower value for long piles in loose sand
p' = average vertical effective stress along the pile
A_s = pile shaft area.

$$Q_p = Nq \times p' \times A_p$$

where

N_q = traditional bearing capacity factor primarily depending on the friction angle of the soil
p' = effective overburden pressure at the pile tip elevation
A_p = pile cross-section area.

Contrary to the static formula given above there are dynamic formulas based on the driving resistance. The dynamic formula mostly recommended in Norway is the formula derived by Prof. N. Janbu at the Norwegian University of Science and Technology, Norway. According to his formula, the bearing capacity is a function of driving energy, material and cross-sectional area of the pile, and final penetration of the pile. To get the design load capacity the ultimate capacity must be divided by a material factor of 1.7–1.9.

The Janbu formula is:

$$Q_u = \frac{2 \times \eta \times W \times H}{s + \sqrt{s^2 + \dfrac{2 \times \eta \times \alpha \times W \times H \times L}{A \times E}}} \times m$$

where

Q_u = pile bearing capacity
W = weight of hammer in kN
H = drop height in m
s = final set per blow in m
L = length of pile in m
A = cross-sectional area of pile in m^2
E = modulus of elasticity for pile material in kPa
η = correction factor, mostly depending on type of hammer, typical variation 0.5–0.9
α = correction factor for load distribution along the pile, typical variation 0.5–0.9
m = equivalent material factor, either 1.4–1.6.

Example If the weight of hammer is 80 kN, the drop height is 1.4 m, the pile length is 40 m, the final set is 0.001 m, the cross-section is 0.0354 m^2, the modulus of elasticity is 2.10 E + 08 kPa, the hammer correction factor is 0.85, the load distribution correction factor is 0.9 and the equivalent material factor is 1.6, the Janbu formula gives a bearing capacity of 4045 kN.

The dynamic formulas cannot be used in silt or clay where pile driving results in high porewater pressures. This type of formula is not relevant for piles driven to bedrock. The evaluation of such piles should be based on the behaviour of the pile after reaching the rock surface. The drop height of the hammer is increased stepwise up to a height giving dynamic stresses in the pile of the same magnitude as the design load. Each step shall be kept until the set per blow comes to a predetermined minimum value, and the sets show a decreasing tendency. The predetermined minimum value should be in the order of 2–5 mm per series of 10 blows. This driving procedure is widely recognized as preventing the tip from sliding on steep rock surfaces, and obtaining the necessary contact area of the pile tip.

When calculating the pile's load-bearing capacity, the steel pipe itself should only be considered as formwork owing to the corrosion risk. The entire axial load on the pile should be taken by the reinforced concrete. As the pipe is emptied prior to reinforcing and concreting, the pile can be regarded as a reinforced concrete pile cast in dry formwork. Usually the pile is reinforced only 2–4 m below sea bottom, depending on soil conditions.

11.4 Lamella berths

Lamella berths can be a good alternative to cell berths when the berth structure will have to accommodate ships before the filling inside the cell structure can be completed. Figure 11.24 shows a lamella berth in principle.

The structure should be designed in such a way that its deadweight alone gives stability which is sufficient to avoid overturning caused by the fill behind the berth. Its deadweight plus the effect of anchoring bolts at the rear end of the lamellas provide its total stability against overturning moments from the fill and the live loads. The dimensions of the bolts should also allow for possible corrosion. In order to increase the deadweight of the structure, the rear part of the platform can be shaped as shown in Fig. 11.24 where a certain amount of fill adds to the stabilizing weight.

Lamella berths are relatively expensive structures that require much diver work and use of heavy formwork and powerful cranes. The lamella berth type should, therefore, not be the first choice if other alternatives are acceptable. If it still provides the best alternative, much thought should be given to finding a construction method that involves a minimum of underwater work. Such a method would imply on-shore prefabrication of formwork and reinforcement for the lamellas in units of maximum allowable size for the available crane capacity. The bottom of the formwork must be shaped according to the rock profile. After placing of the form, including the reinforcement, it must be anchored to resist waves, wind and current until the lamellas have been concreted and permanently bound together by the platform structure.

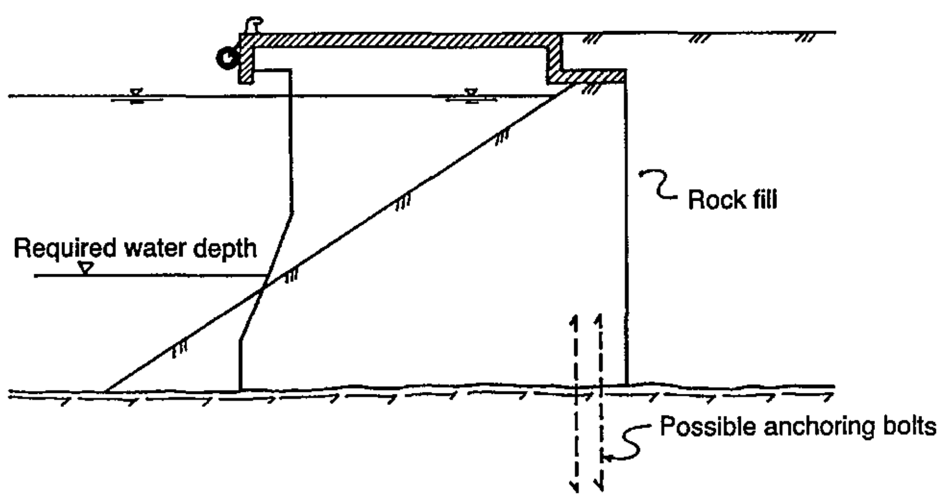


Fig. 11.24. Lamella berth

11.5 Open berth slabs

The berth slab must be designed in such a way that the vertical loads are transmitted safely by way of the beams to the columns, piles or the lamellas, and the horizontal loads from ships' impacts, moorings, etc. to those parts of the structure that are meant to absorb them. A typical cross-section of an open Norwegian berth type is shown in Fig. 11.25.

In the first reinforced concrete berth structures built in Norway, high and narrow rectangular beams supported the slabs. After 10–15 years of use these structures showed deterioration in the form of corrosion of the reinforcement at the bottom of the beams and subsequent cracking and scaling-off of the concrete covering the reinforcement. The slabs had usually not deteriorated to the same extent. The reasons for this are many: the most important factors were that the beams came too close to the sea level, and were densely reinforced, had too small concrete cover and generally were more difficult to concrete satisfactorily.

To avoid these disadvantages, beamless slabs were built which proved very durable. However, this type of structure is more costly to build, particularly due to the necessary formwork support system and, nowadays, one is back to the slab/beam type of structure. The difference from the old slab and beam structures is that the modern quay structures have low beams, broad and trapezoidal cross-sections, as shown in Fig. 11.26. Thus most of the disadvantages experienced with the old beams are avoided. The trapezoid in Norway is now the normal shape of the beam cross-section in open berth structures.

It generally applies that the formwork should have an over-height at mid-span corresponding to the deflection due to the deadweight of formwork and concrete. The top of the slab should, with a view to the practical use of the berth, lie about 50 cm above the highest high water observed and it must also be put at a level high enough to permit the beams under it to be concreted in a dry form.

The load-bearing capacity of columns of 80 cm diameter and more is seldom fully utilized, and rather the dimensions of the beams and the slabs themselves determine the lengths of beam and slab spans. To avoid very high formwork costs, the slab spans are usually about 6–7 m and the beam spans about 8–10 m. Preferably only one span length and beam cross-section for each beam should be maintained throughout the berth structure. This makes it possible to use the same formwork over and over again for many spans. Possible differences in loadings and/or moments should be reflected by variations of the amounts of reinforcement rather than by different span lengths or beam cross-section. The formwork cost usually amounts to only 10–15 per cent of the total

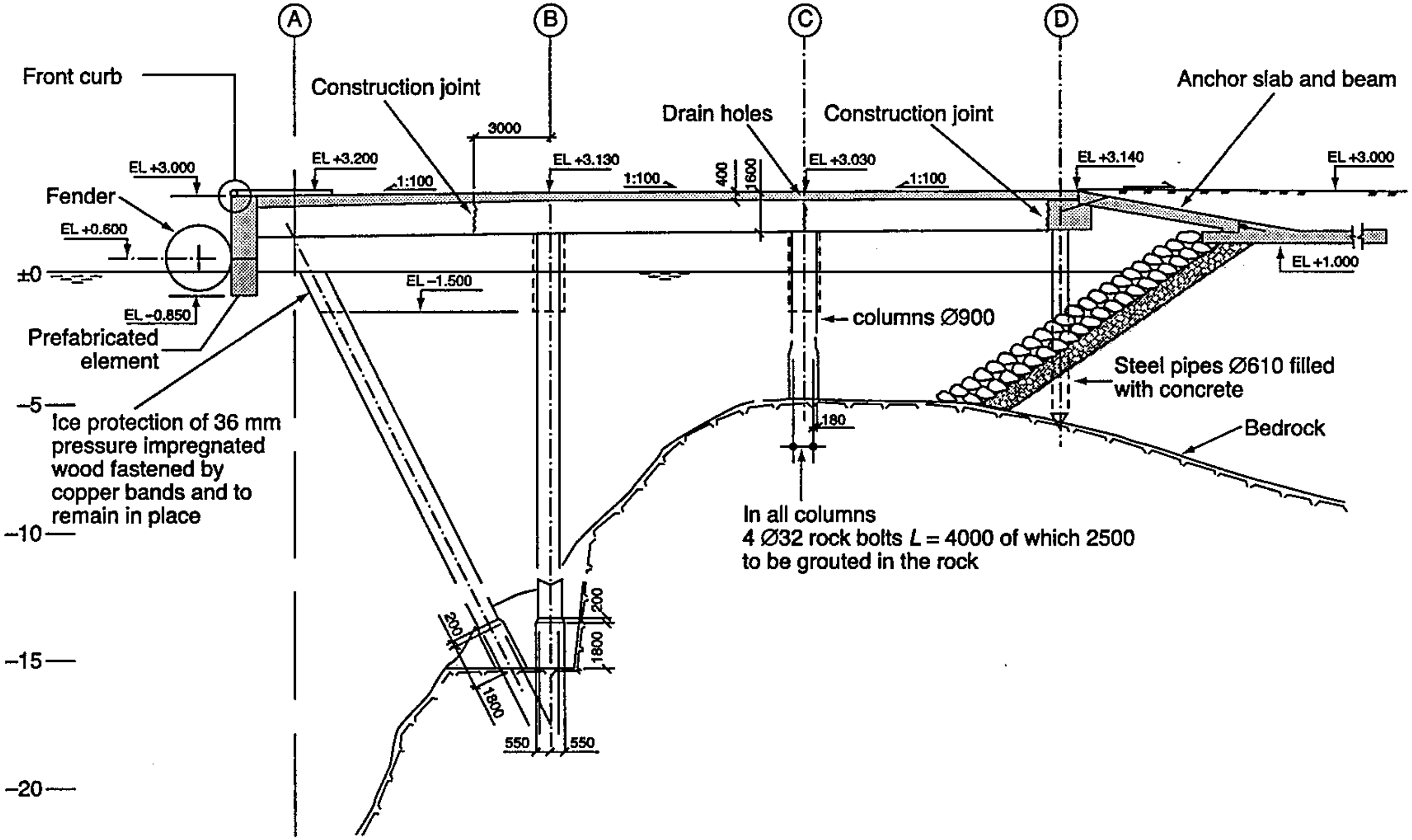


Fig. 11.25. Cross-section of an open berth

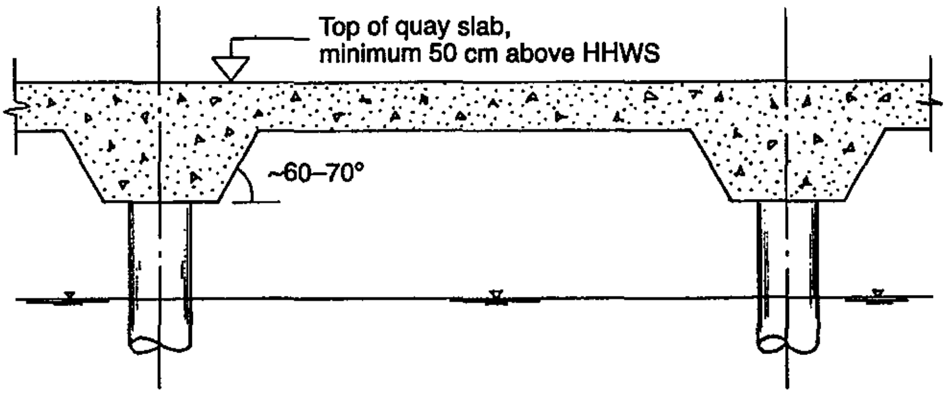


Fig. 11.26. Cross-section of berth slab and beams

construction cost, but, nevertheless, the planning and building of the formwork are very important for a successful construction of the works.

Three different types of rational berth formwork system for construction are described in the following sections.

11.5.1 Jacket form system

The use of jacket form systems is based on the concreting of beams and slab in one operation. The forms are supported by steel beams resting on column brackets, as shown in principle in Figs 11.27 and 11.28. The support for the beam formwork can be either of concrete brackets on the columns as shown in Fig. 11.28 or of steel brackets as shown in principle in Fig. 11.29 and in detail in Fig. 11.30.

The system requires great precision in the placing of columns and supporting brackets on the columns or piles. If a column is somewhat out of place problems are likely to arise. Rafts are used for the installation, moving and dismantling of the formwork, and the system is therefore best suited where the under-platform clearance is large or there are high tidal variations.

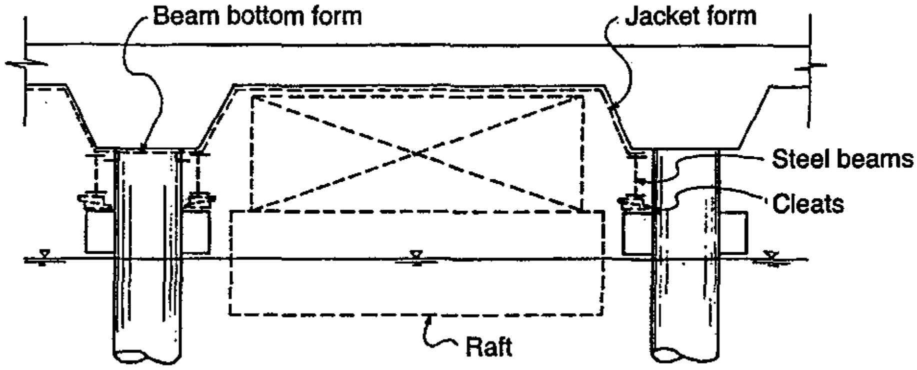


Fig. 11.27. Jacket forms

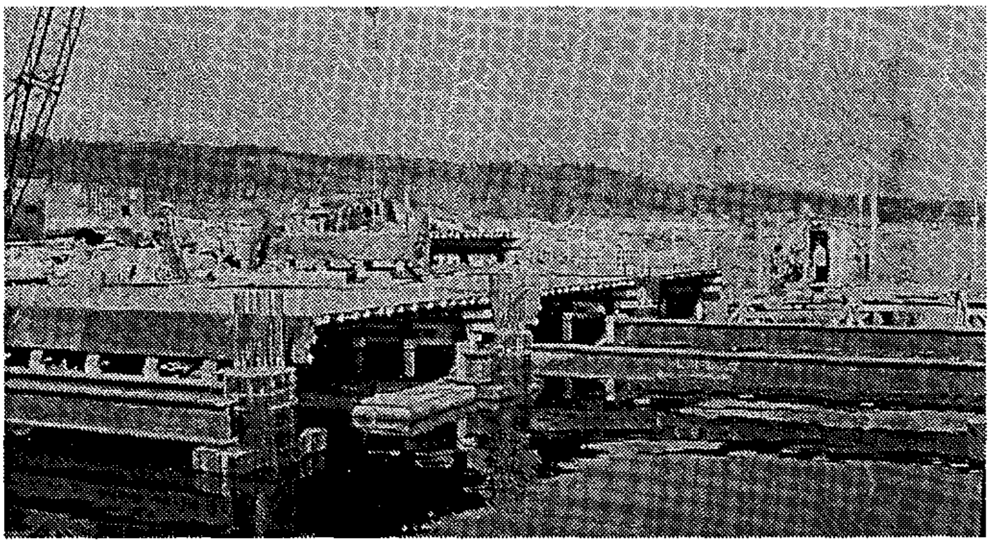


Fig. 11.28. Jacket forms as shown in detail

The dismantling of the formwork should be given great attention at the planning stage because access from above is not possible after concreting. All details must be such as to allow the lowering of the formwork onto the raft without difficulties. The raft must be designed for lifting and lowering by pumping of water in and out of it or by using the tidal variations.

11.5.2 Girder systems

Use of a girder system for the slab span concreting is as follows. First, the beams are formed and concreted up to the bottom of the deck slab. The formwork for the beams is either supported as shown in Fig. 11.29, or

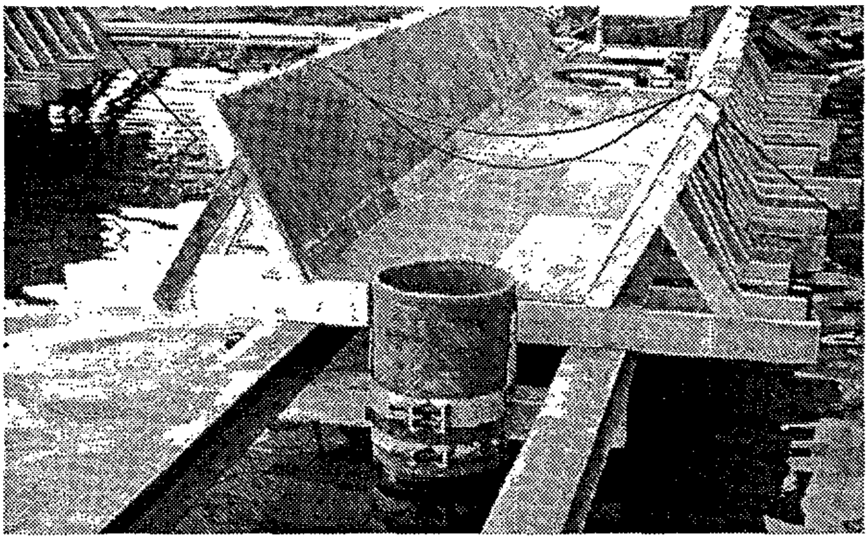


Fig. 11.29. Support by steel brackets

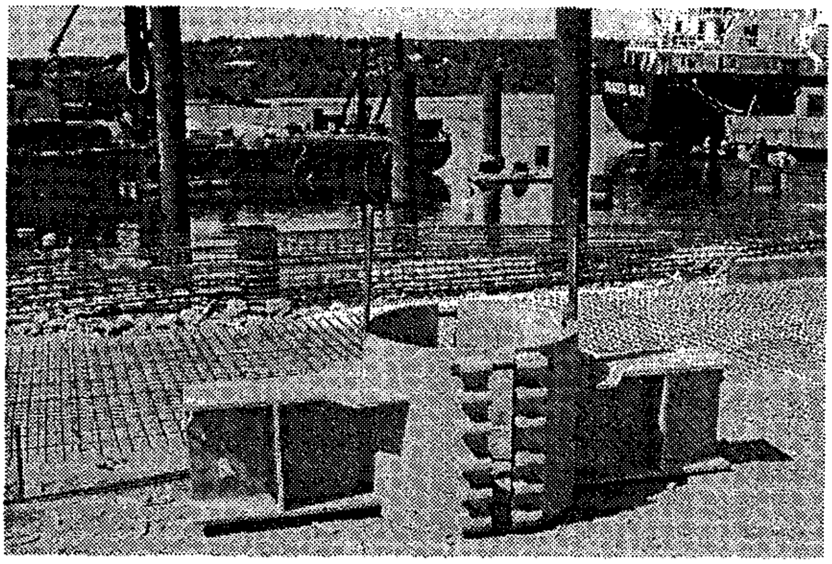


Fig. 11.30: The steel brackets as shown in detail

the beam support can be hung from the column itself, as shown in Fig. 11.31.

Then girders spanning between the concreted beams are installed to serve as supports for the slab span formwork, as shown in Figs 11.32 and 11.33.

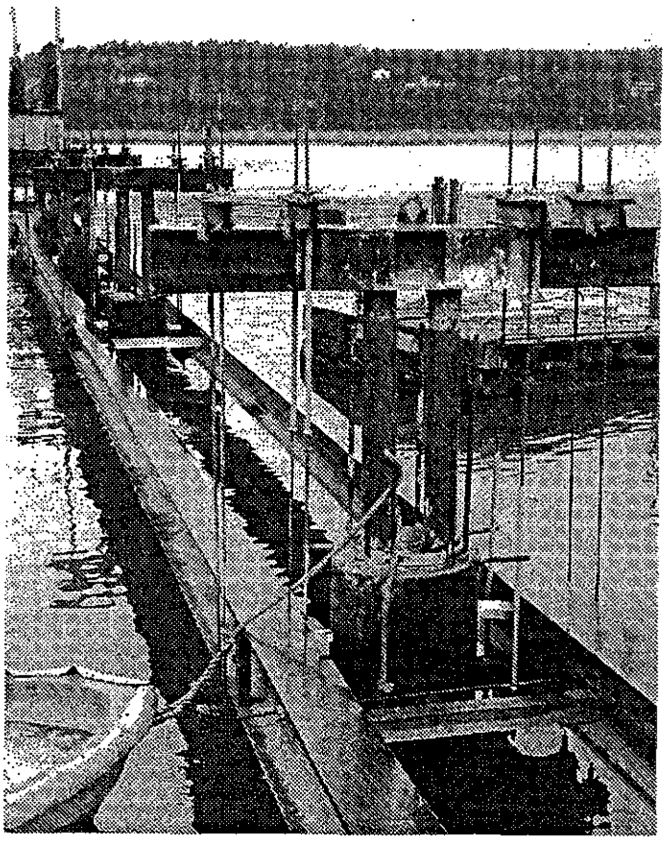


Fig. 11.31. Support by hanging from the top of the columns

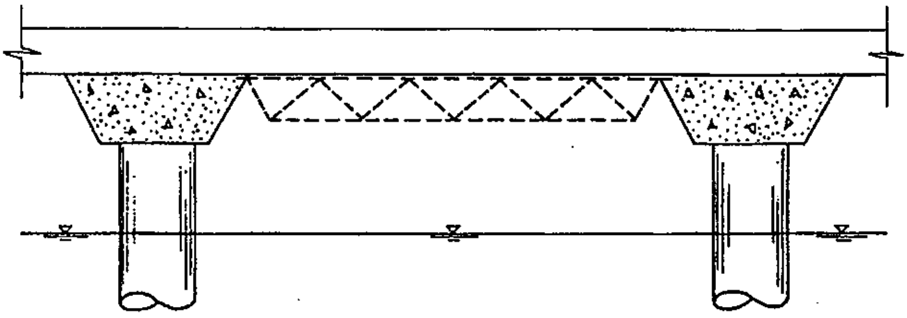


Fig. 11.32. Girders

The concrete beams are formed in the same way as when jacket forms are used, but the supporting steel beams for the concrete beam itself will have smaller dimensions. This is a very flexible system and very useful if, for instance, adjustments due to out-of-place columns are needed. Particularly in pile berth construction this flexibility is very welcome because the precision in placing or driving the piles is less than in the placing of columns. Figure 11.34 shows the beam during the installation of the reinforcement.

Figure 11.35 shows the finished concrete beams before the use of either girder system or precast deck slab elements.

11.5.3 Precast concrete element system

Figure 11.36 shows the following five different types of precast or prefabricated elements generally used for a berth structure: anchor

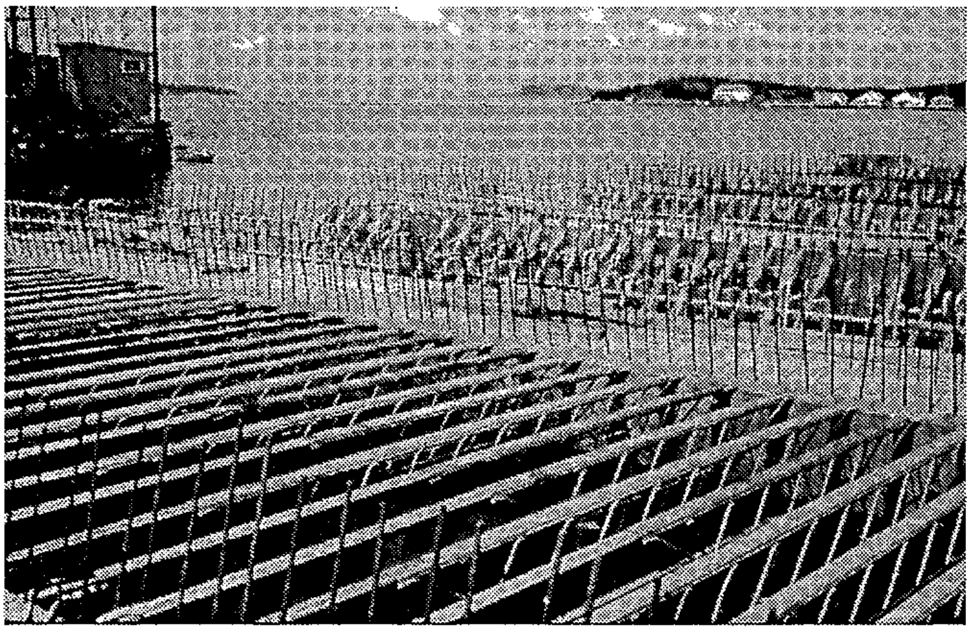


Fig. 11.33. Girders spanning between beams

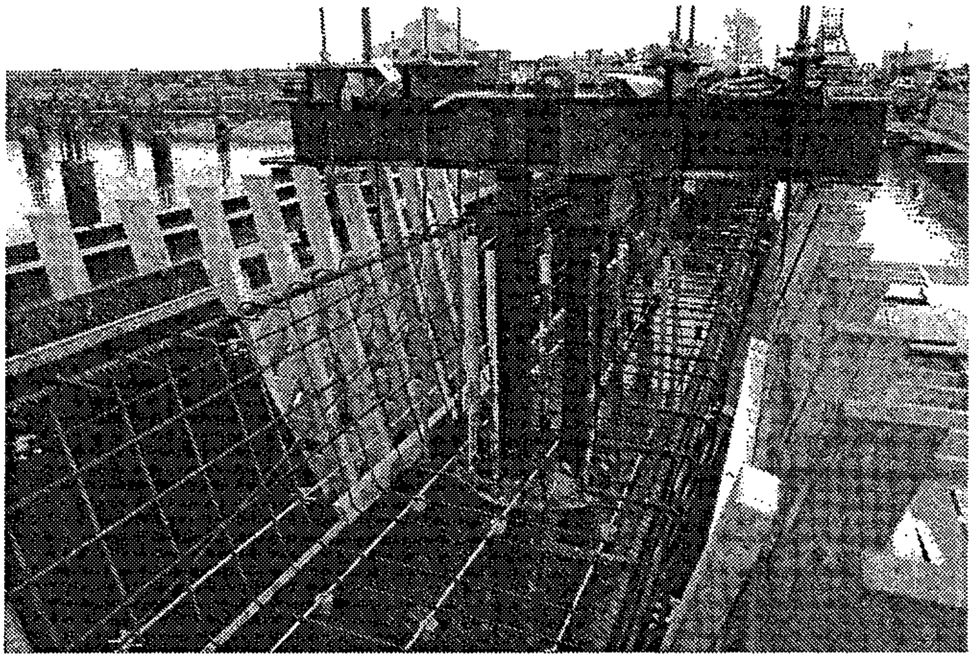


Fig. 11.34. The beam during the installation of the reinforcement with the beam formwork hanging from the column

slab element, settlement slab element, deck slab element, front beam element and front wall element. A berth properly built with precast elements will be of the same quality and have the same design lifetime as an ordinary monolithically built berth.

The elements can be either non-prestressed reinforced concrete elements or prestressed concrete elements. The elements are installed

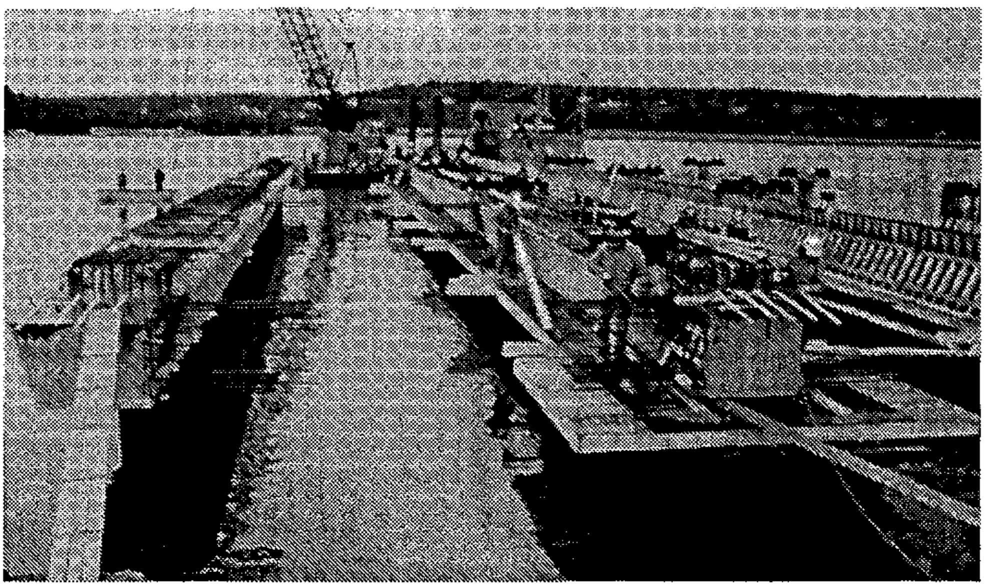


Fig. 11.35. The beams before the installation of the girder system or the deck slab elements

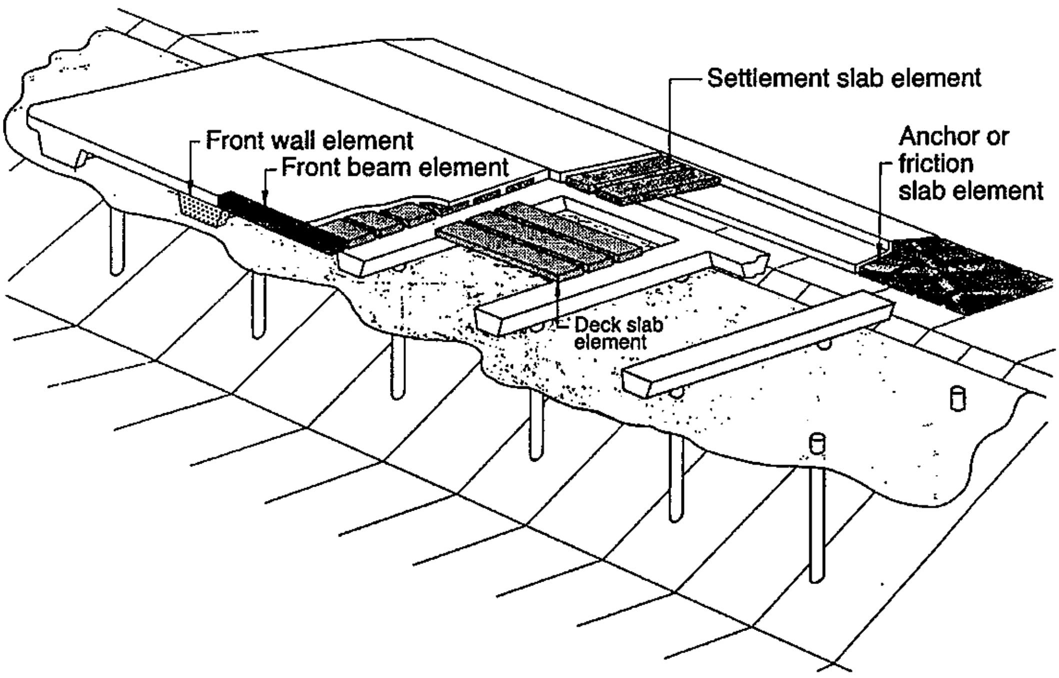


Fig. 11.36. Berth with non-prestressed elements

by using a mobile or floating crane. A berth built with prestressed elements (pretensioned or posttensioned) will not have the same monolithic strength against impact from loads as a berth built of non-prestressed elements, as shown in Fig. 11.37. One should bear in mind that prestressed elements themselves can hardly be repaired

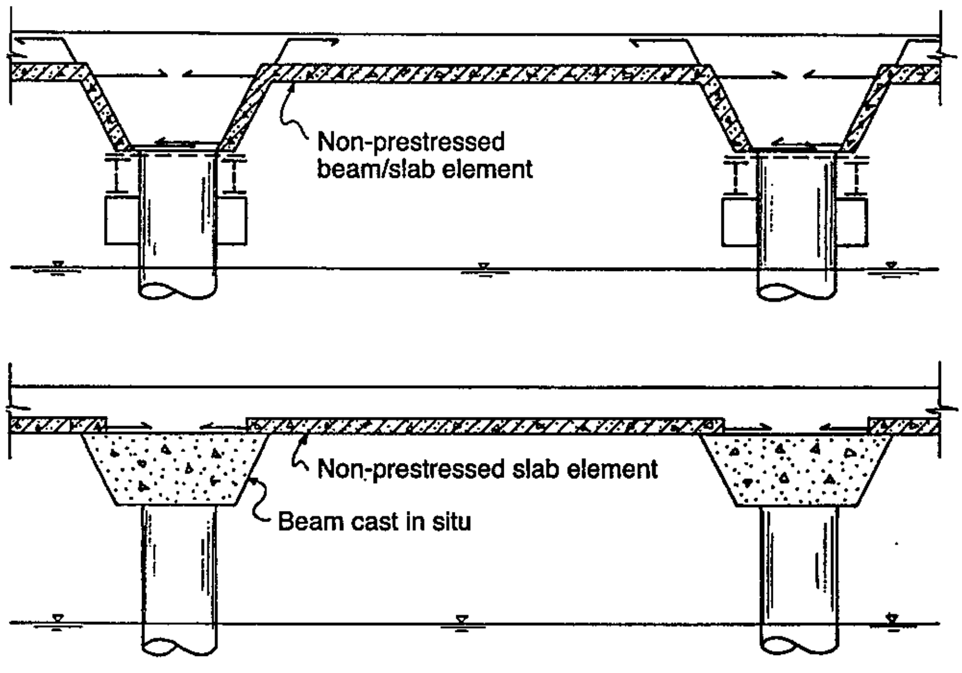


Fig. 11.37. Non-prestressed elements

after possible damage or deterioration of the concrete or corrosion of the reinforcement.

Precast or prefabricated non-prestressed concrete elements for construction of berth structures are commonplace worldwide. The prefabrication of elements is an effective measure for reducing both the time of construction and costs. The advantages of precasting or prefabrication are as follows:

(a) reduction of construction time
(b) minimizing costly formwork and cast in-place concrete
(c) generally less dependent on the weather condition
(d) good quality of the concrete produced.

The disadvantages of precasting or prefabrication are:

(a) sensitive to the weather condition during the installation process
(b) usually necessary to have large floating-crane capacity
(c) small tolerances for installation.

The use of elements in berth construction can be advantageous due to shorter construction time since the elements can be made simultaneously with other works (e.g. installation of piles, columns, etc.), better uniform concrete quality in the elements, etc.

Figure 11.38 shows the non-prestessed beam and slab elements installed at the formwork.

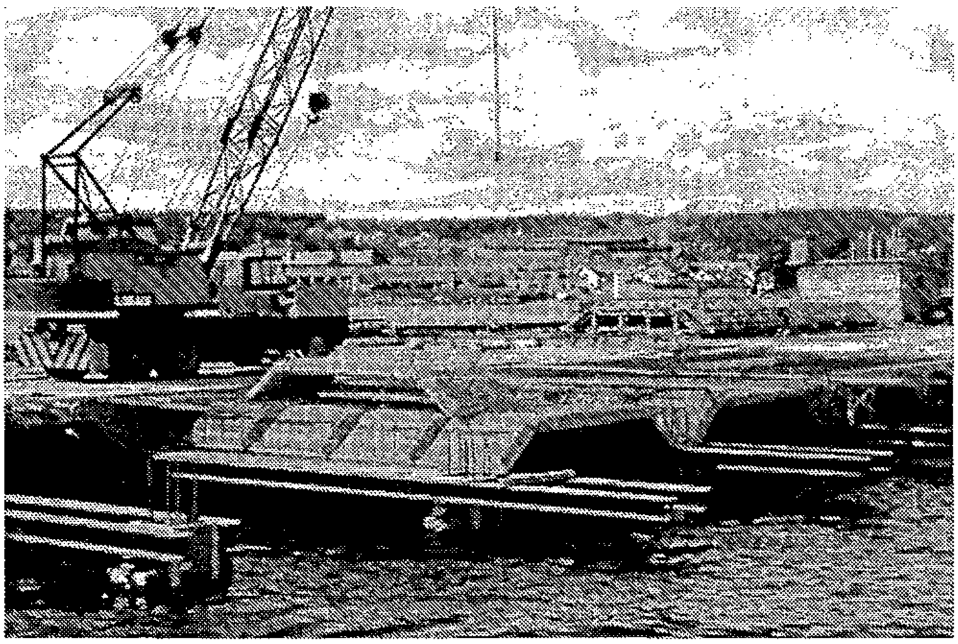


Fig. 11.38. Beam side and deck slab element

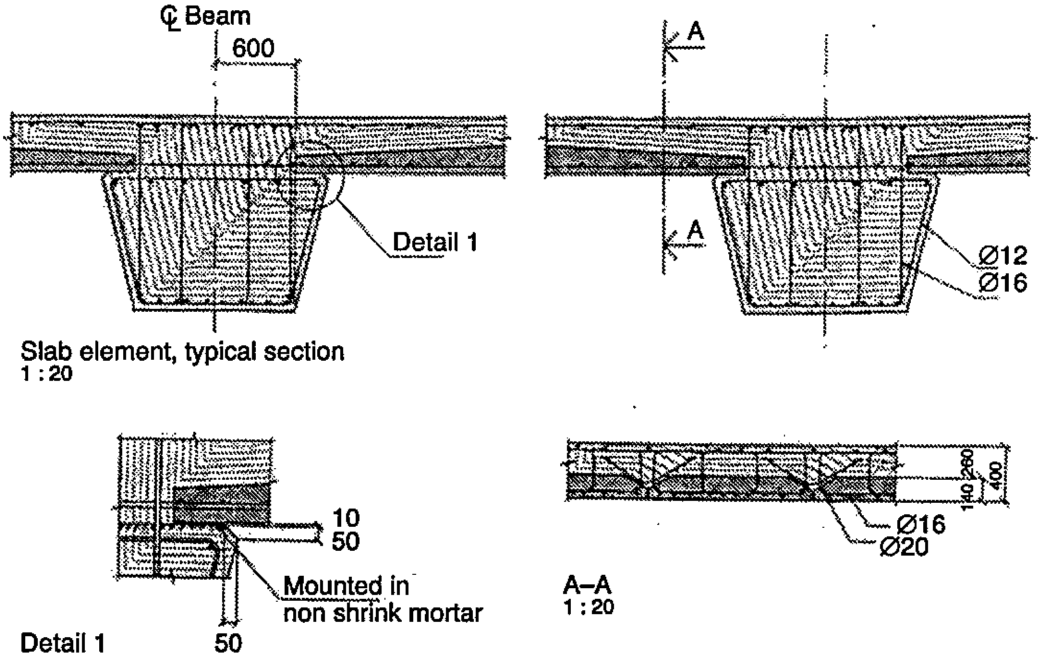


Fig. 11.39. Cross-section of the beam and the deck elements with the reinforcement

Figures 11.39 and 11.40 show a cross-section of the beam and the deck elements with the reinforcement and the support on the beams.

In many cases it could be economical to prefabricate the berth beams, as shown in Fig. 11.41. The slabs between the prefabricated beams are usually deck slab elements, as shown in Fig. 11.40.

Figure 11.42 shows the lifting of the deck slab element into position by a mobile crane with 40 kN lifting capacity and it being placed on the berth beams before the mounting of the top reinforcement in the deck slab. Figure 11.43 shows an overview before all the deck elements have been installed and Fig. 11.44 after the deck element has been installed.

For the crude-oil jetty shown in Fig. 11.45 the loading platform slab was constructed by use of small deck slab elements as shown in Fig. 11.44. The crude-oil jetty was designed for berthing of 300 000 dwt oil tankers.

Figure 11.46 shows a combined beam and slab element with a weight of approximately 4000 kN being lifted into final position over the steel piles. With this system the beams and the deck slab are installed in one operation.

Between the deck slab and the settlement slab, and between the settlement slab and the anchor slab element, as shown in Figs 11.47 and 11.48, there must be a hinge so that the anchor or friction slab can absorb any possible settlement in the soil beneath the anchor

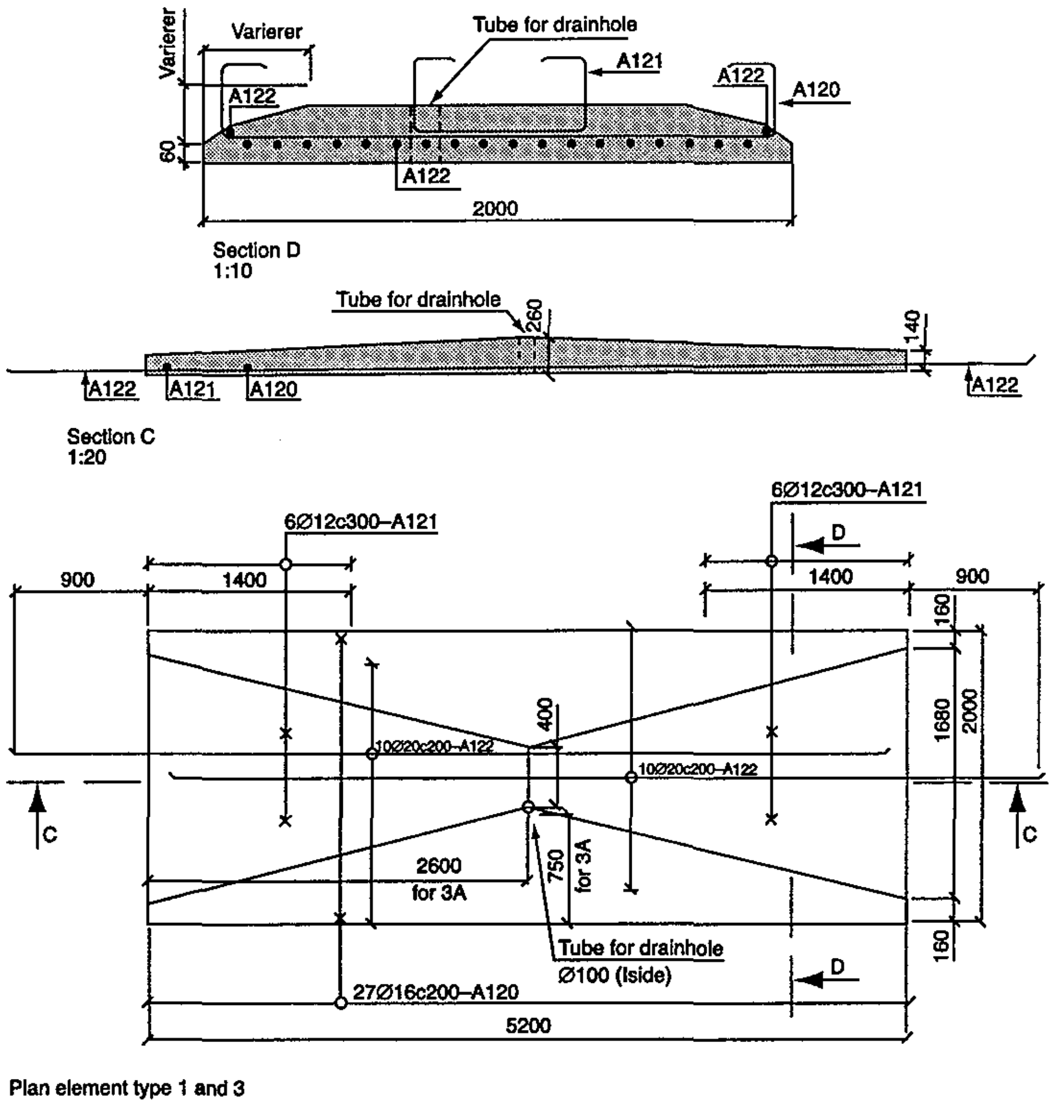


Fig. 11.40. Reinforcement in the deck slab elements

slab. The hinge is usually designed for a settlement of at least 50–60 cm. In Figs 11.48 and 11.49 details of the hinge between the deck and the settlement slab, and the settlement slab and the anchor slab are shown. In Fig. 11.50 the hinge reinforcement between the deck and settlement slab is shown.

Figure 11.51 shows the settlement slab between the finished berth deck and the anchor slab being lifted into position.

The settlement slab can also be constructed as a large finished concreted element. Figure 11.52 shows a finished concrete element with a total weight of approximately 3000 kN being lifted into position between the berth slab and anchor slab.

As can be seen from Fig. 11.36, most structural elements in a berth or quay structure can be prefabricated. The choice of precast elements in

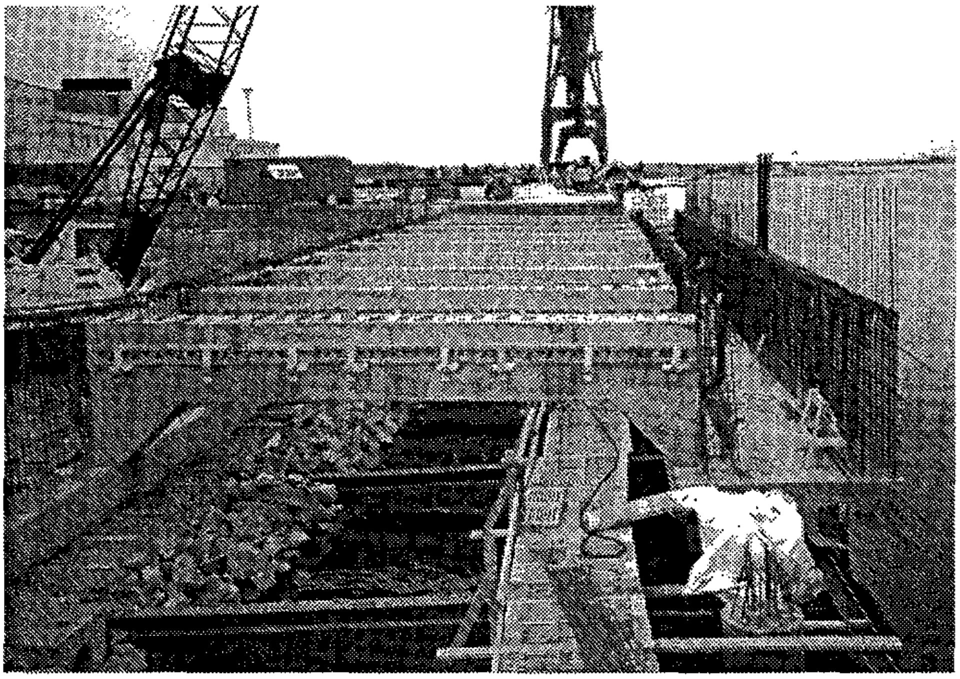


Fig. 11.41. Prefabricated beam elements

the superstructure will mainly be based on economic considerations. With the increasing availability of equipment for transportation and for heavy lifts, prefabrication is now a common procedure. The advantages of prefabrication will be reduction of construction time, more efficient quality control, standardized design and construction, and it will be favourable in places where the land space available for construction is very small. The disadvantages of prefabrication will be, among other things, availability of suitable lifting equipment, small tolerances and stability during the construction period.

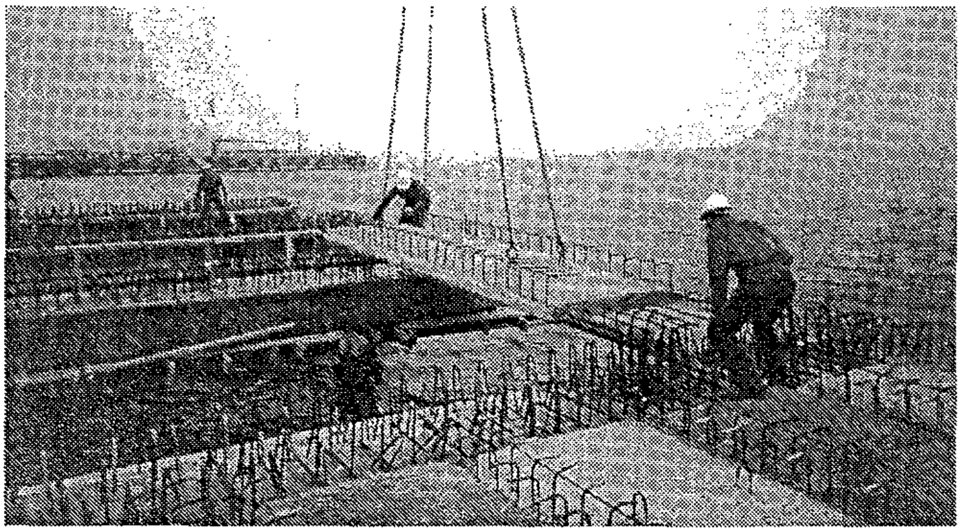


Fig. 11.42. Deck slab element lifted into position by a crane of 40 kN capacity

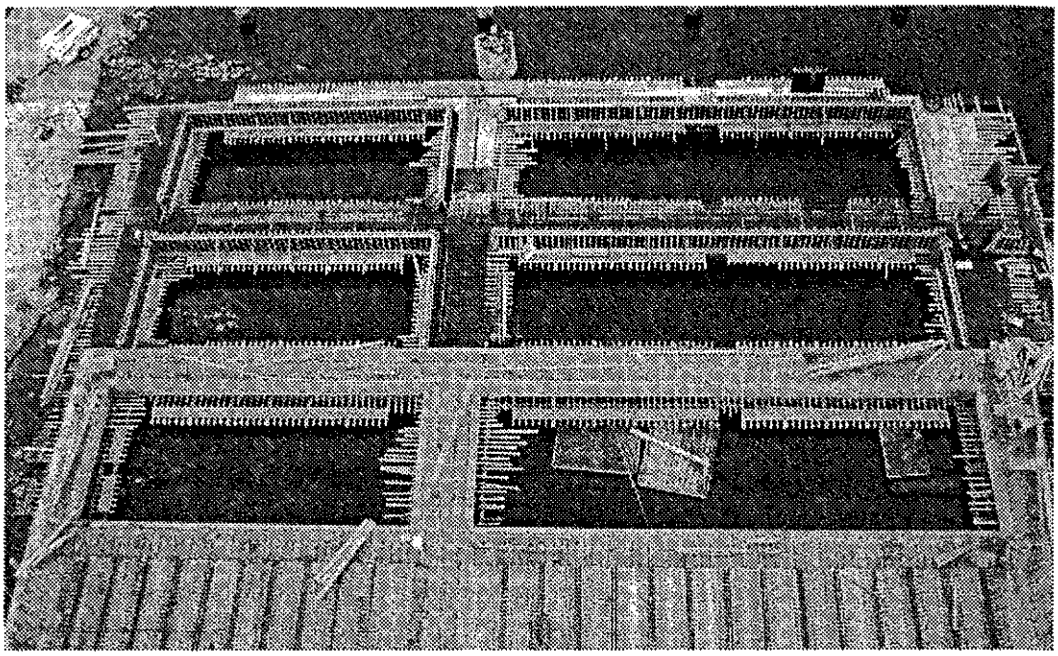


Fig. 11.43. An overview before all the deck elements have been installed

Which of these systems is chosen is a question that has to be solved in each separate case. Important factors are then the expected lifetime of the berth structure, durability requirements and the contractor's experience and equipment.

The construction of the berth front with suspension of the formworks from the deck and the reinforcement is shown in Fig. 11.53. The

Fig. 11.44. Deck slab element

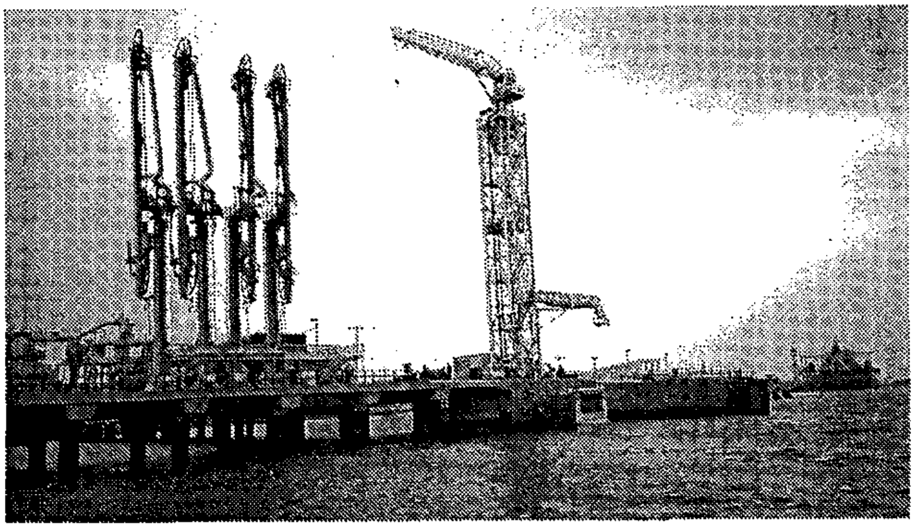


Fig. 11.45. Crude oil jetty

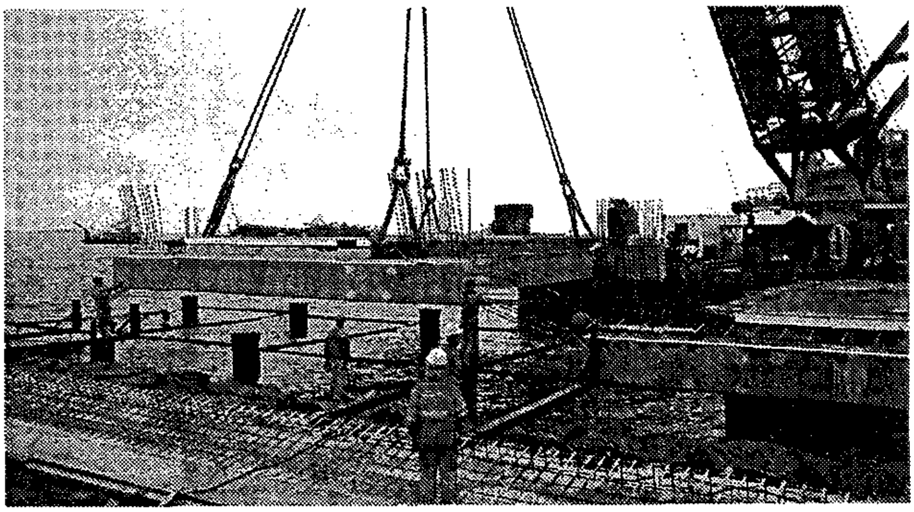


Fig. 11.46. Large beam and slab element of approximately 4000 kN lifted into final position

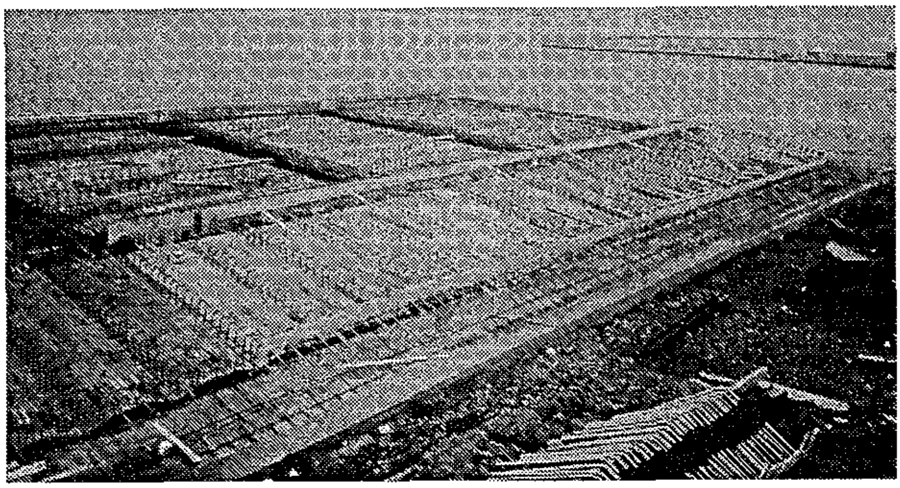


Fig. 11.47. Deck slab element and settlement slab element

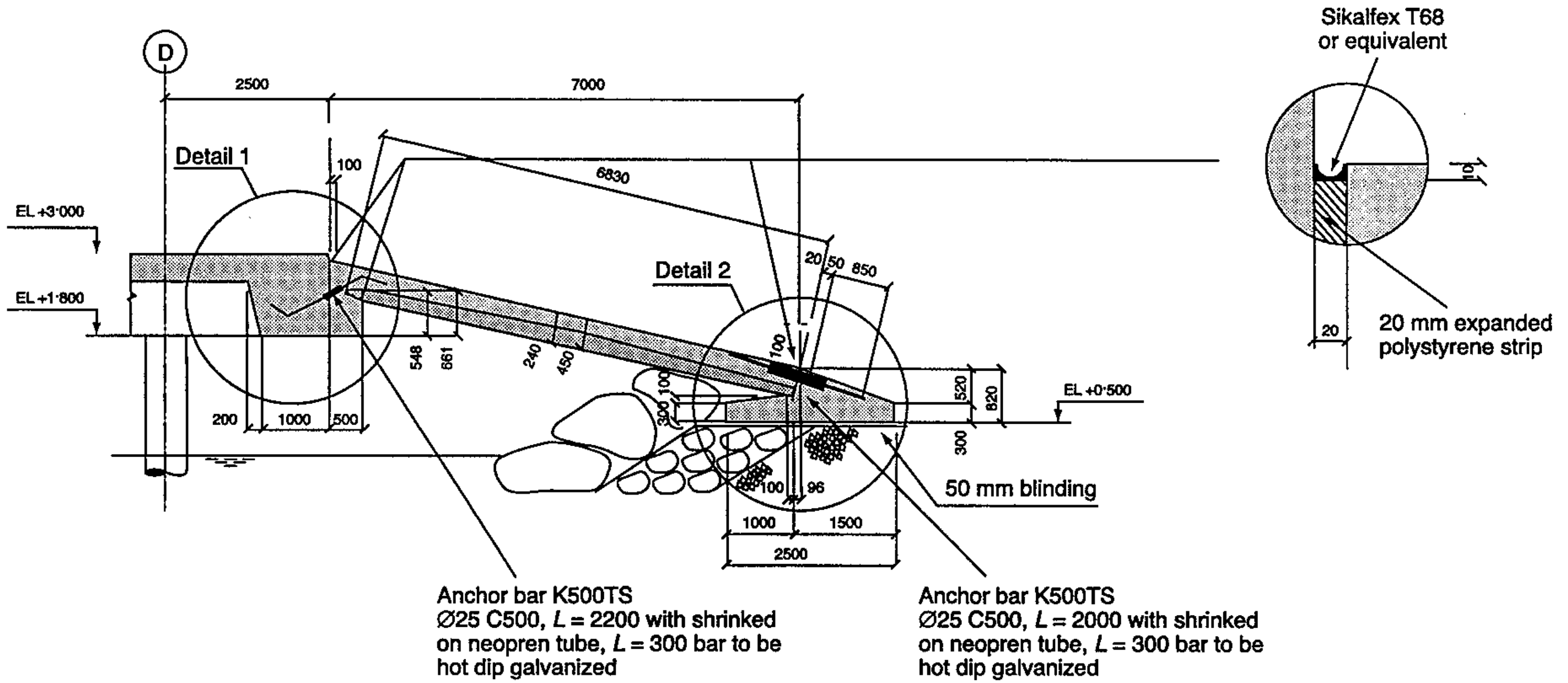


Fig. 11.48. The settlement slab

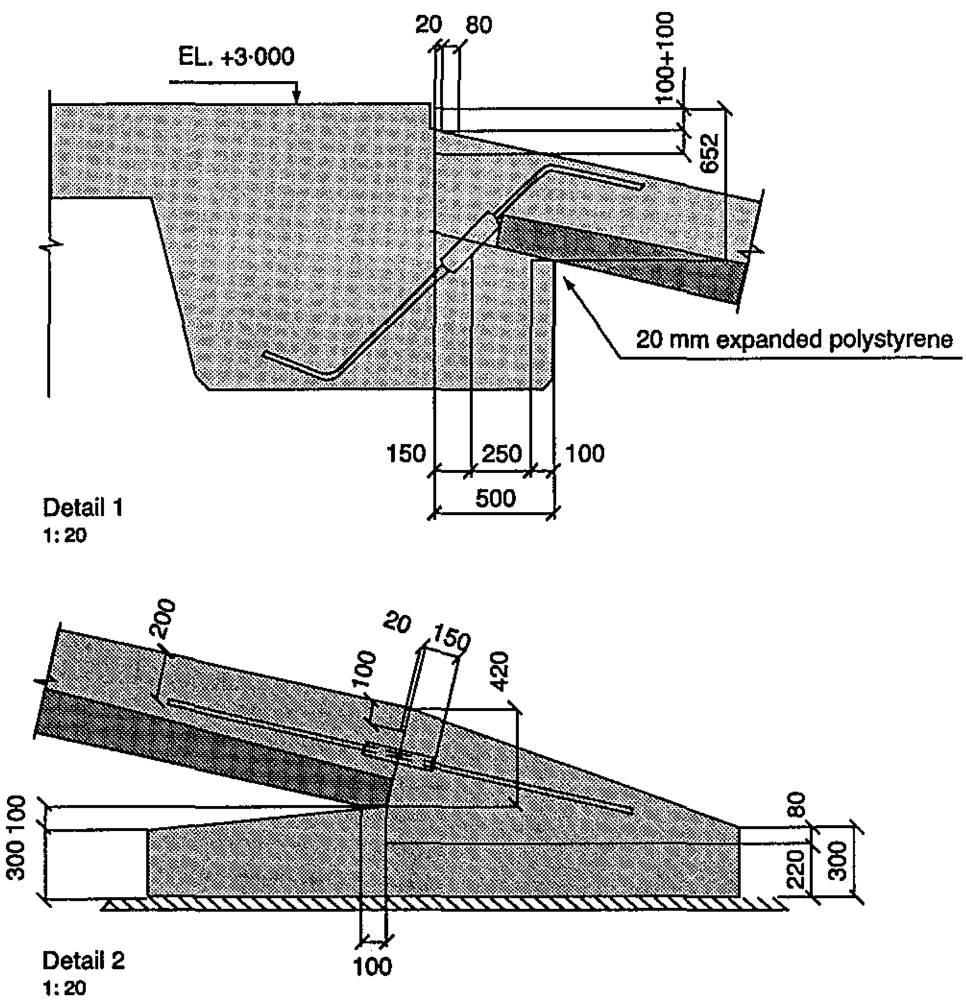


Fig. 11.49. Detail of the hinge between deck and settlement slab and anchor slab

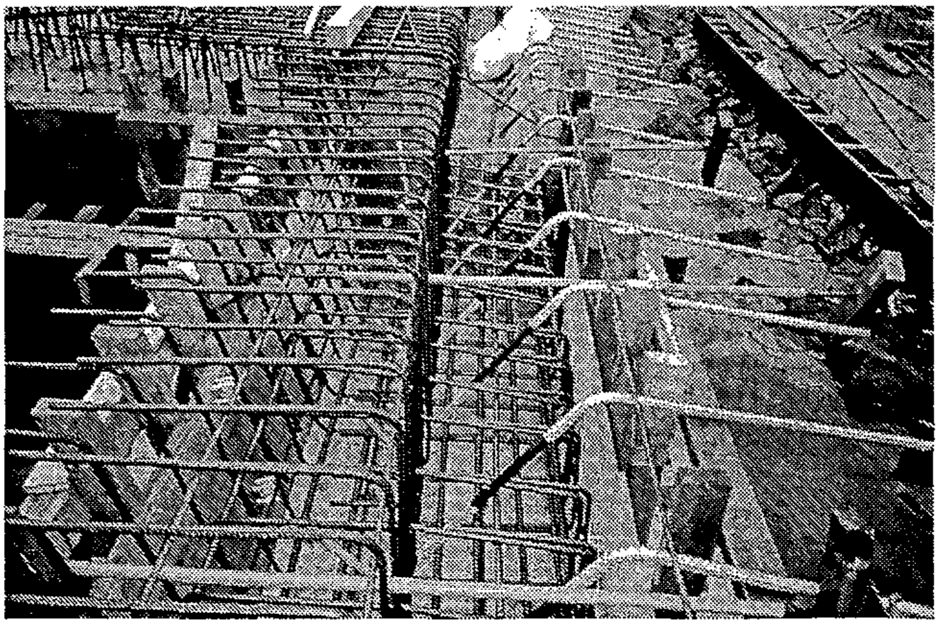


Fig. 11.50. Detail of rear beam and the hinge reinforcement to the settlement slab

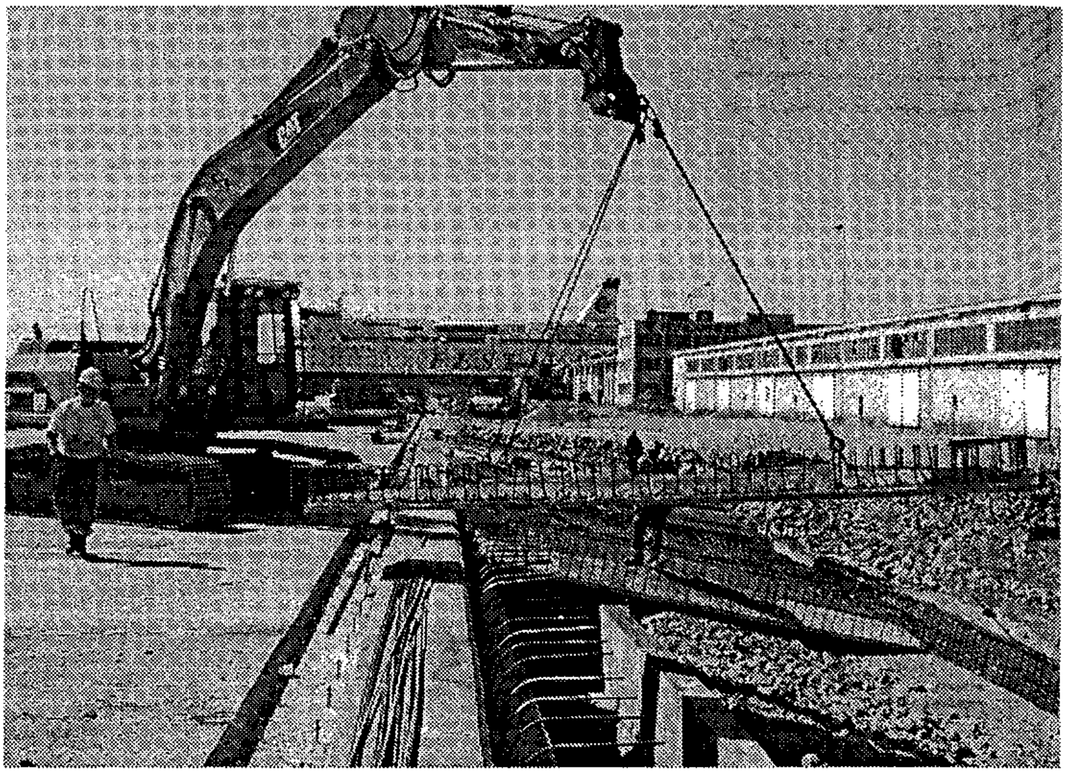


Fig. 11.51. Settlement slab element lifted into position

finished berth front equipped with fenders, front curb and rescue ladders is shown in Fig. 11.54.

A common element berth is shown in cross-section in Fig. 11.55 and with details shown in Fig. 11.56. On top of the concrete or steel piles,

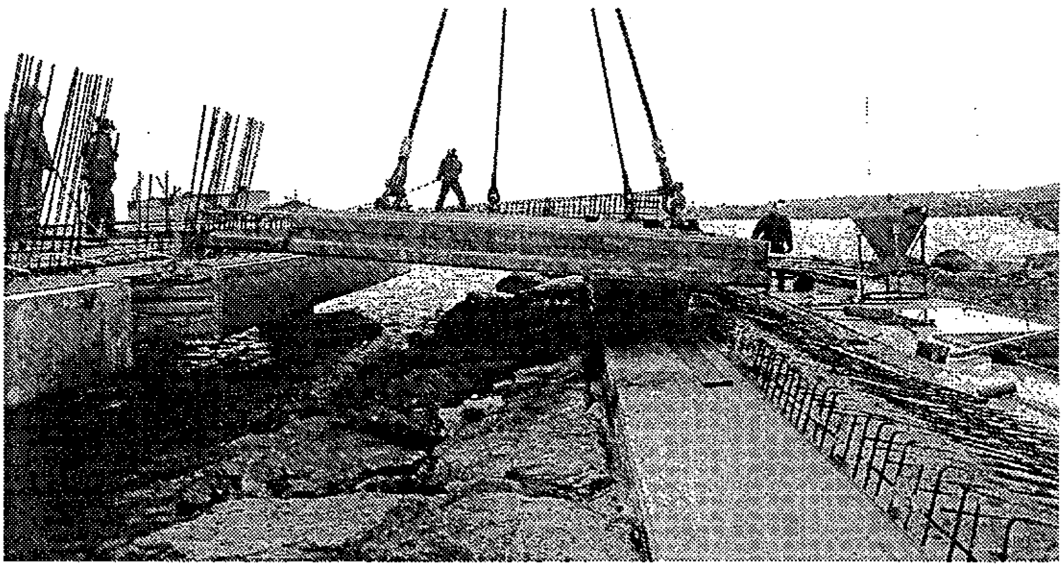


Fig. 11.52. Large settlement element of approximately 3000 kN lifted into final position

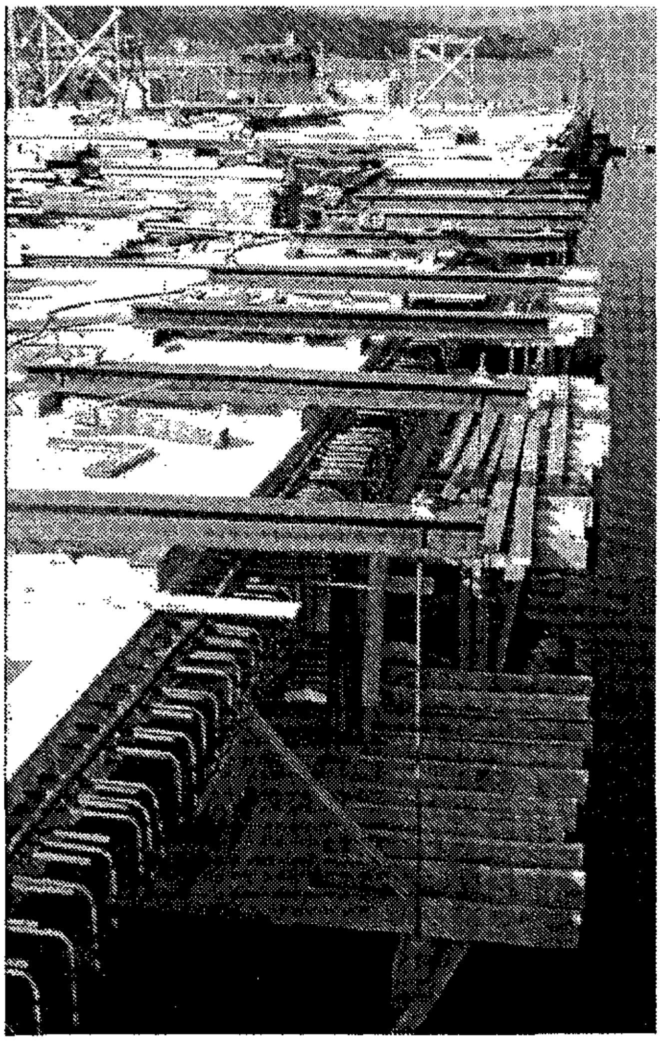


Fig. 11.53. Construction of the berth front

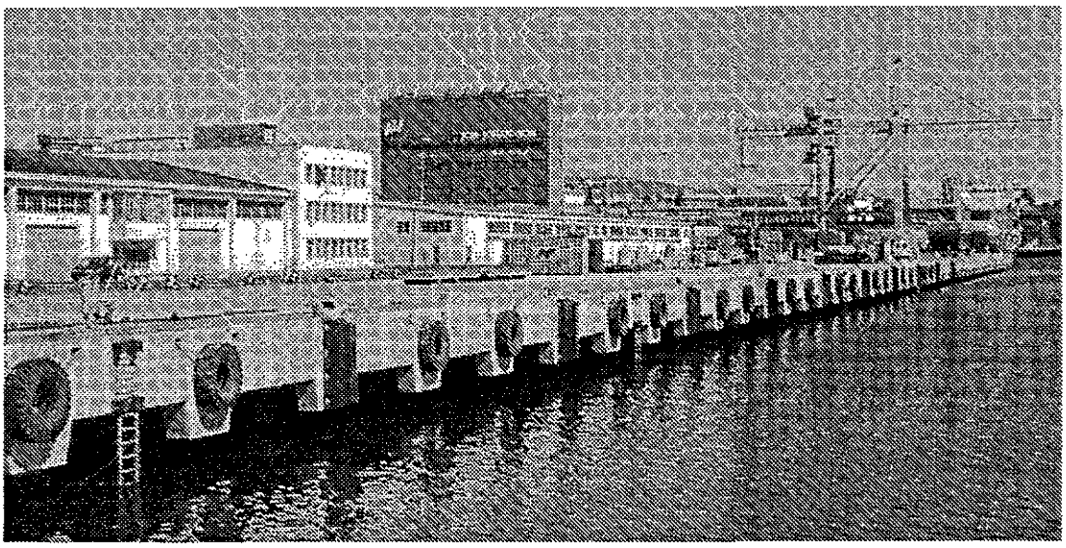


Fig. 11.54. The finished berth front

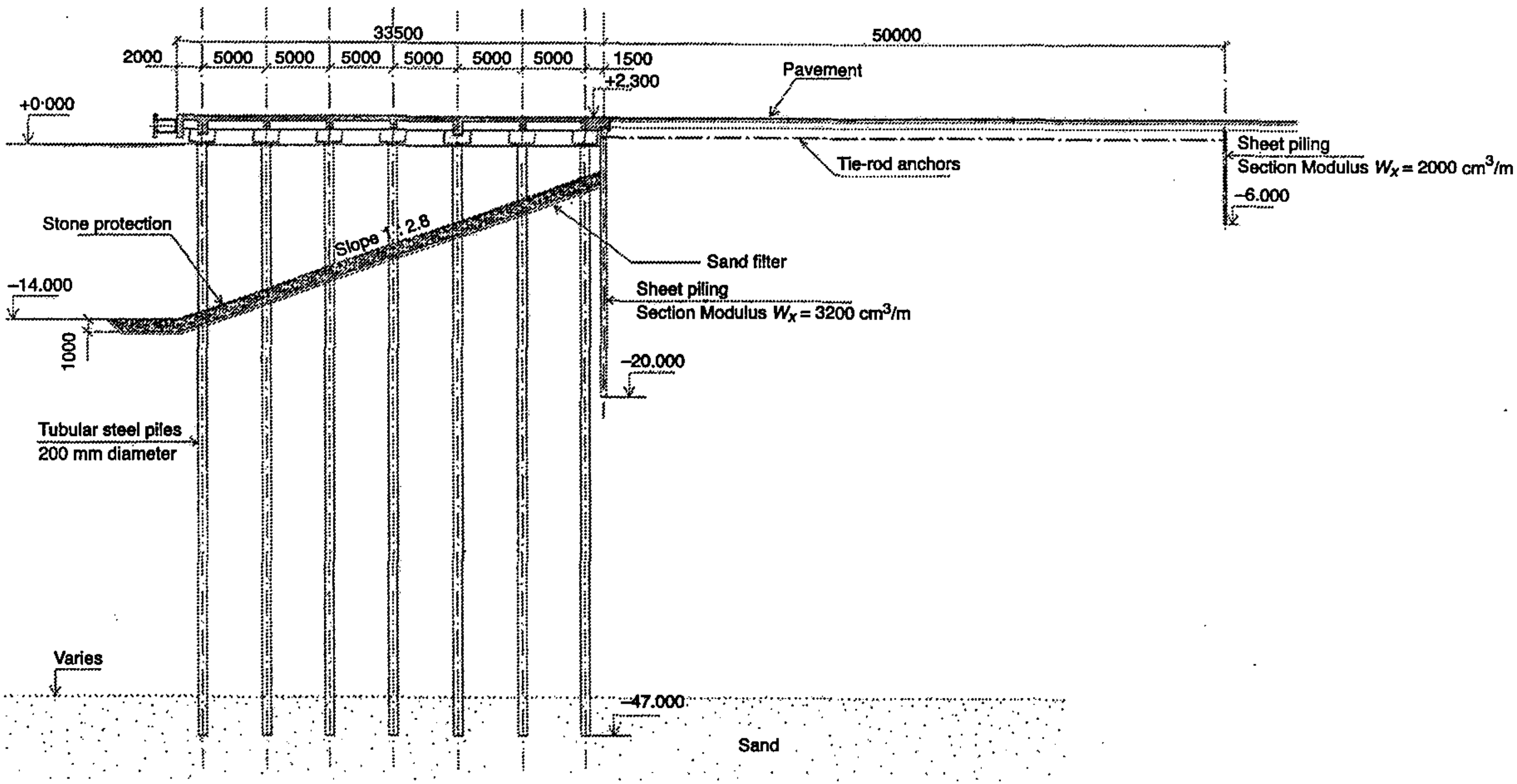


Fig. 11.55. *Cross-section of an open element berth*

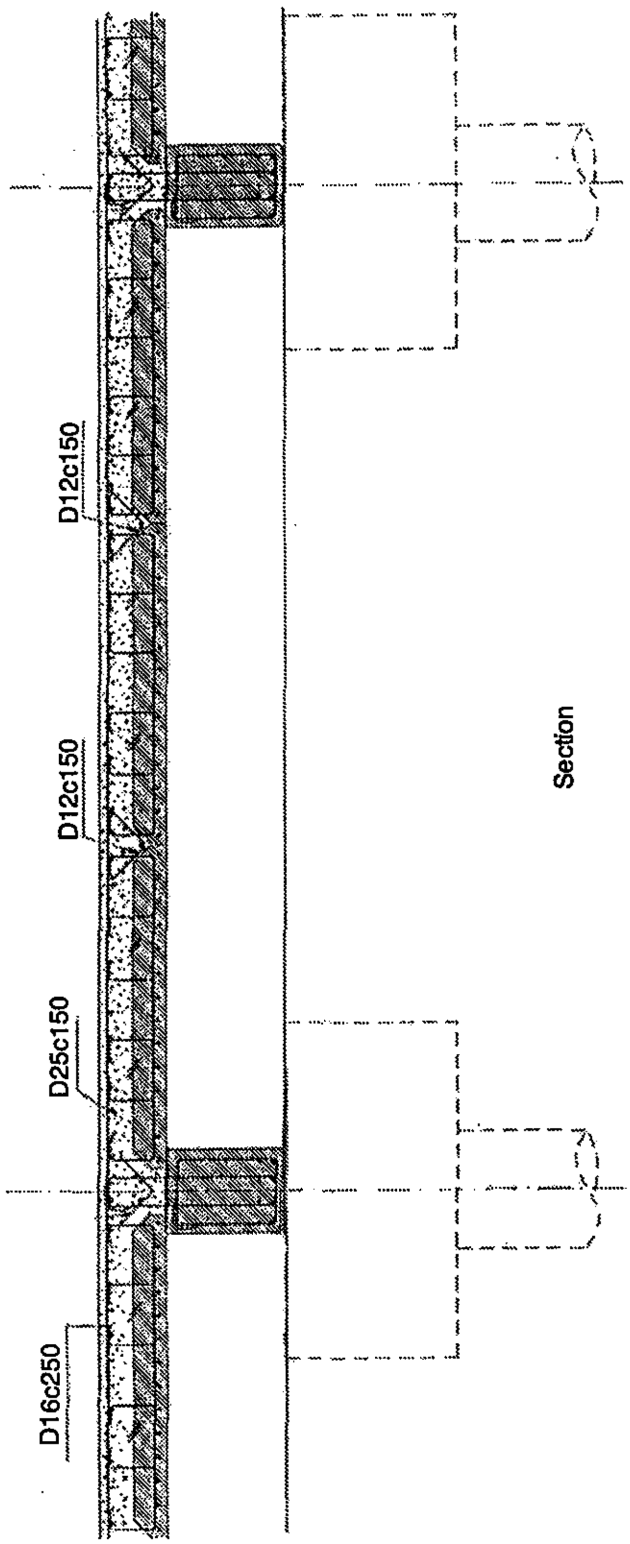


Fig. 11.56. Element details

reinforced concrete cap units are installed for supporting the prefabricated concrete beams and deck-slab elements. The final deck slab is concreted after the top slab reinforcement has been installed.

To resist traffic wearing on the slab it should be provided with a protective pavement on top. If made of concrete, this top layer can either be placed together with the concreting of the slab itself thus constituting a 3–5 cm additional part of the cast in situ monolithic slab. It can also be made separately, after the curing of the slab, as an 8–10 cm reinforced top slab. Generally, the first method is recommended but if very difficult weather conditions can be expected during the concreting the top layer should be placed at a later stage under more favourable weather conditions.

The maximum use of prefabricated berth elements may be adopted to achieve an earlier completion date, but the size and weight of the different concrete elements have to be within the handling capacity of the available crane.

Further reading

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12

Berth details

12.1 General

When working out the berth slab details, consideration must be taken to the installation of berth equipment such as fenders, bollards, sockets for power and telephone, water outlets, etc.

The planning of supply facilities at the berth structure and at the terminal area should include at least the following supply facilities:

(a) lighting
(b) electric power
(c) potable and raw water.

And the following discharge facilities:

(a) water drainage
(b) sewage disposal
(c) oil and fuel interceptors.

12.2 Lighting

The berth structure and access roads and terminal area should be equipped with sufficient suitable lighting during all berth operations and at night as a defence against crime. The following are therefore recommended:

(a) Lighting during terminal operation and during loading and unloading of the ship should be 100 lux.
(b) Lighting for security of the port area should be 30 lux.

12.3 Electric power supply

Only underground cables for low and high voltage supply systems to the port installations, crane installations, lighting etc. should be used. The earth cover of the supply system should be approximately 0.8–1.0 m. The power connection points along the berth front should be at intervals of approximately 50–200 m depending on the type of berth activities.

12.4 Potable and raw water supply

To safeguard the delivery of potable and raw water supply to the port or terminal area, at least two delivery lines are required, independent of each other, to each port or terminal section. Hydrants should be installed at approximately 100–200 m intervals. In cold areas, the water pipelines system should be placed with sufficient earth cover to be protected against frost.

12.5 Water drainage system

The drainage system for berth structures and terminal areas can be divided into the following systems:

The **open system.** The top surface of the berth structure and the terminal area should be designed to allow spray from the waves and rainwater to be drained away directly to the harbour. For areas where differential settlements can be anticipated the cross falls should be as high as 1:40. For surface areas with no settlements risk, the cross fall should be between 1:60 and 1:100. The drainage system, through the berth deck, is shown in Fig. 12.1.

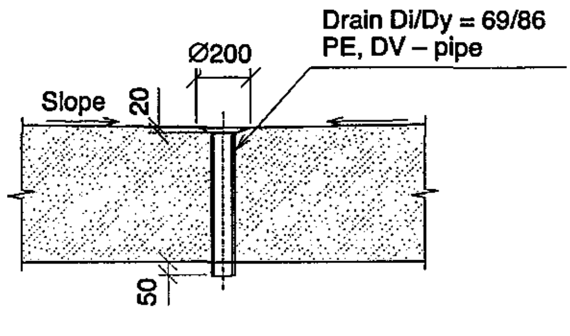


Fig. 12.1. Drainage detail

The **closed system**: where the water can or may be polluted, for example, with possible oil spillage during the loading of oil products at an oil berth, the surface water must be collected to a separate drainage system for treatment.

12.6 Sewage disposal

Any sewage disposal in the port area should be fed through a special pipe sewerage system to the municipal system or to a dedicated treatment facility.

12.7 Oil and fuel interceptors

All oil and fuel waste should be collected in special interceptors.

12.8 Access ladders

Access ladders should be placed at 50 m intervals along the front of the berth structure. In order to be accessible from the water, the ladder must extend down to 1 m below the LAT. In order to give the ladders sufficient strength, the ladders should be designed for a horizontal and vertical load of 1.0 kN/m. Figure 12.2 shows a flexible ladder hanging from the front steel rail kerb and a stabilizing weight of an old rubber tyre filled with concrete.

12.9 Handrails and guardrails

Handrails should be provided on both sides of walkways and on part of the berth structure itself if they do not interrupt cargo handling or mooring arrangements for ships. The top of the handrail should be at least 1.0 m above the berth deck and walkway elevation as shown in Fig. 12.3.

Along, for example, an access bridge out to an oil berth structure or along the terminal area against the waterfront guardrails, as shown in Fig. 12.4, should be installed.

12.10 Kerbs

Around the berth edges kerbs should be provided to prevent, for example, trucks from sliding into the water. The kerbs should be at

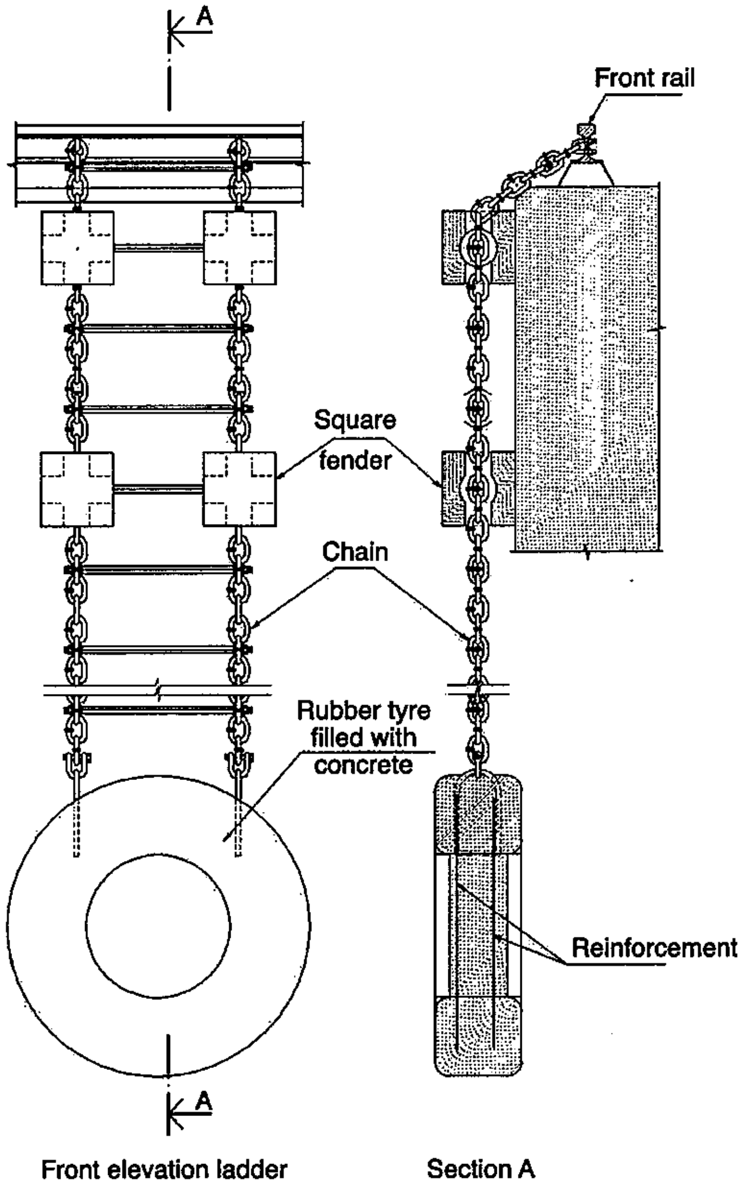
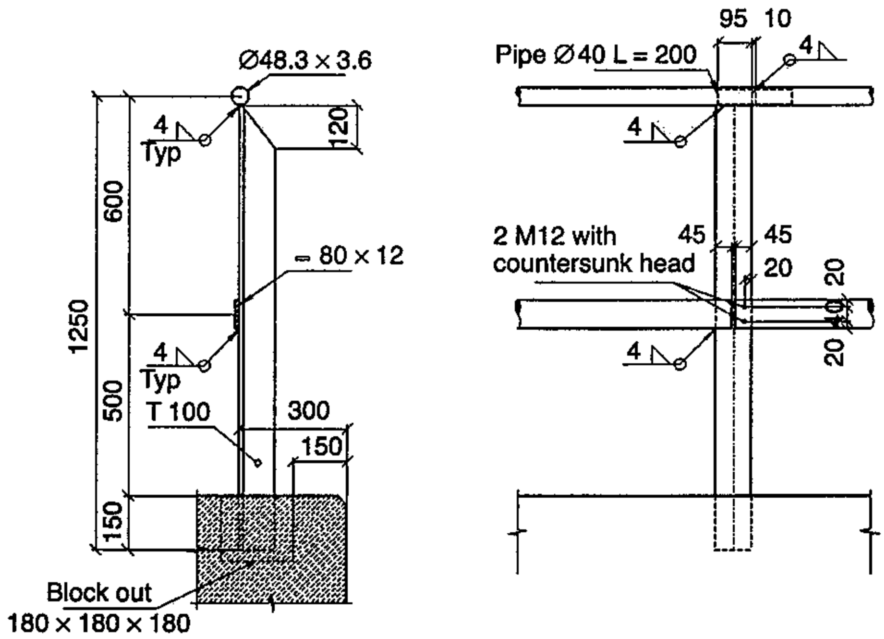


Fig. 12.2. Flexible ladder

least 200 mm high. The kerbs can be either of concrete, as shown in Fig. 12.5 or constructed from used rails as shown in Fig. 12.6.

12.11 Lifesaving equipment

Lifesaving equipment should be installed on all berth structures and especially jetty heads. It is recommended that chains be suspended at the seaward side between the ladders. The chains should be extended to 1.0 m below the LAT. Lifebuoys with approximately 30 m buoyant line should be installed along the berth structure at 50 m intervals.



Section typical handrail

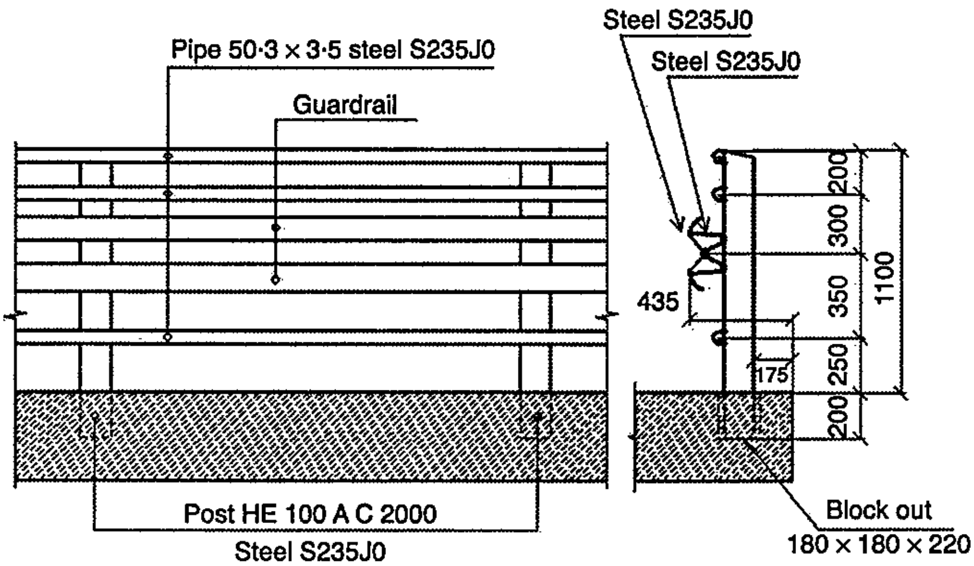
Typical joint

Fig. 12.3. Typical handrail

12.12 Pavements

12.12.1 General

A durable pavement area with high performance is vital to container and port terminal operations. Nowadays there are different kinds of area pavements and the most common types are: asphalt, cast concrete



Elevation typical guardrail

Section guardrail

Fig. 12.4. Typical guardrail

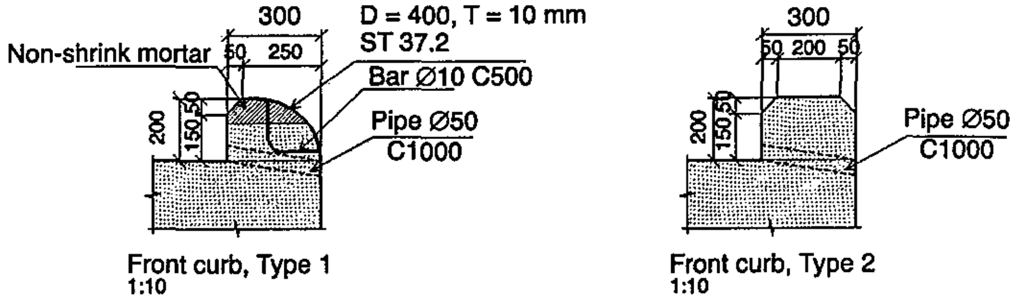


Fig. 12.5. Concrete kerb

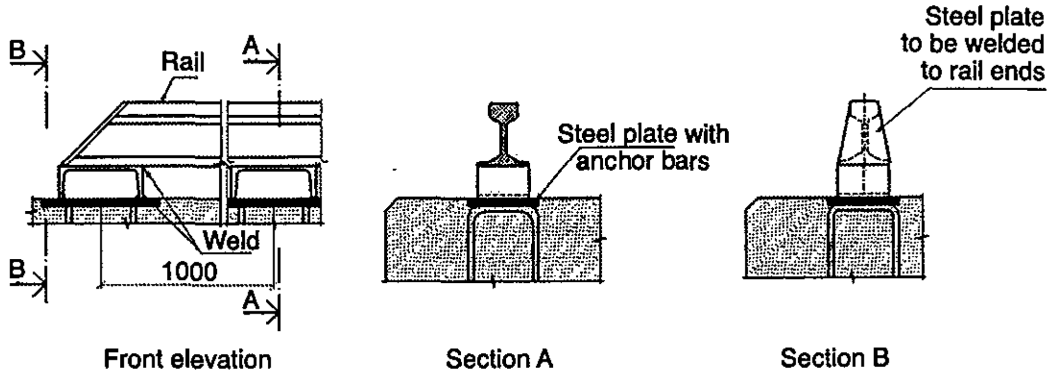


Fig. 12.6. Rail kerb

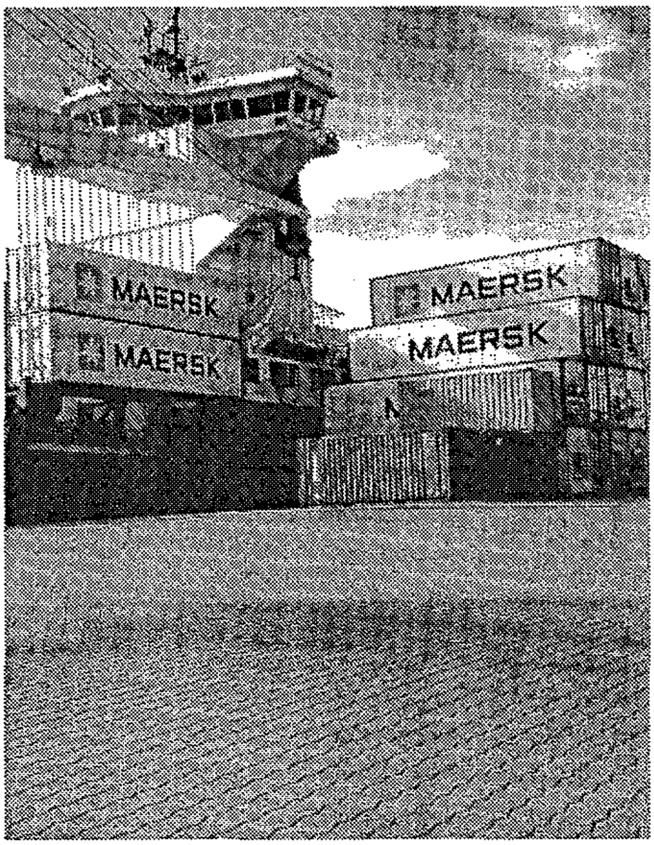


Fig. 12.7. Concrete block paver area

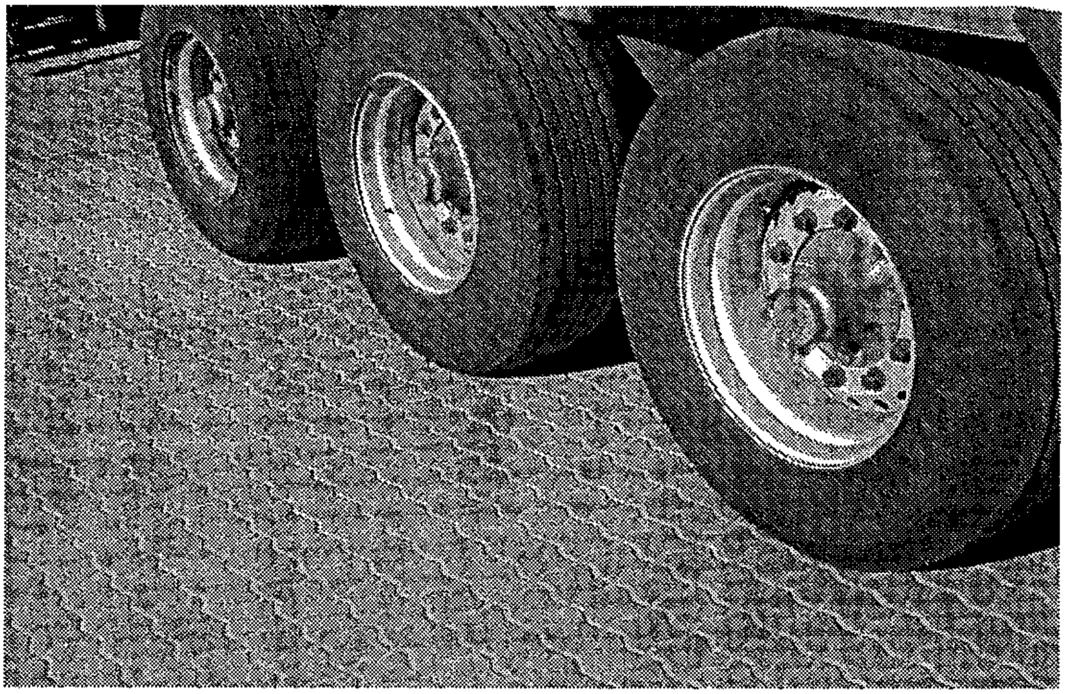


Fig. 12.8. Detail of concrete paver

and concrete block pavers. Figure 12.7 shows a concrete block paver area.

Area pavements of concrete block pavers have proved to be beneficial at areas where heavy equipment is used, such as large fork-lifts. Figure 12.8 shows detail of concrete pavers. The geometric shape and quality of the pavers are of vital importance regarding the pavement's performance. Due to the various methods of engineering design and traditional choice of materials around the world, the following is a general guideline for the design of different base-courses depending on the subgrade. The general recommendations given in this chapter are applicable for heavy duty areas in the harbours. Correctly constructed pavements have the following advantages:

(a) good performance
(b) economical
(c) low maintenance
(d) long durability.

The first block paver of concrete was invented around 1880, and nowadays there are more than 250 different types of concrete pavers. There are an increasing numbers of harbours worldwide where the pavement is carried out in concrete blocks. One of the first projects was carried out at one of the worlds' largest container terminals in Rotterdam in 1965, and is still in service.

12.12.2 The construction components

Base-course (subbase and base)

The complete base-course consists of different material layers, as shown in the figures below. The required dimension and materials used in the different layers will depend on the subgrade condition and the estimated traffic and loads. Within economical limits, best-quality materials should be used. In the following three cases the design loads are a live load of 100 kN/m^2 and an axle load of 1000 kN.

The design of the base-course for a concrete pavement should be of a conservative manner. The base-course for asphalt pavement should be in accordance with each country's national standards. Only minor changes to top of the base are required, which are tight elevation tolerances for the top of the base, and the use of materials of small grading to prevent the bedding sand escaping into the base. The following guidelines and recommendations should generally be followed for the construction of the base-course:

(a) Base-courses should be designed and constructed as for asphalt pavements according to national standards or by the use of specific computer programs.
(b) Mechanical stabilized base/subbase: material in base to be constructed with crushed rock grading approximately 0–30 mm (max. 0–60 mm). Thickness of this layer should be 100–150 mm. When use of subbase: material to be approximately 0–60/200 mm.
(c) Other materials for base: asphalt or cement stabilized.
(d) Elevation tolerance: to comply with the thickness requirements for the thin bedding layer, the tolerance of the upper base should be approximately ± 10 mm for maximum stability.
(e) Materials should meet the quality requirements in the national standards.

Where the subgrade soil condition is very good and has a California Bearing Ratio (CBR) of approximately 25 per cent, the construction and use of the materials should be as shown in Fig. 12.9 and Table 12.1. The total base-course, as shown, will be approximately 45 cm.

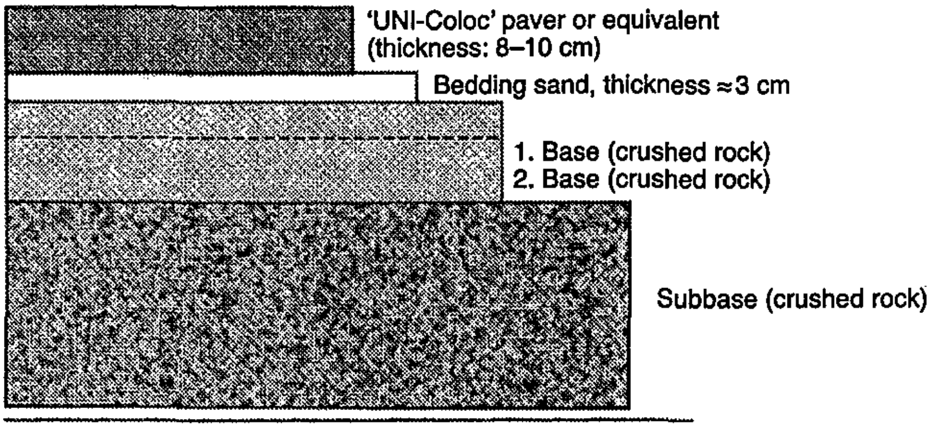


Fig. 12.9. Typical construction for subgrade of CBR ≈ 25 per cent or more

Where the subgrade soil condition is **moderate** and has a CBR of approximately 10 per cent, the construction and use of the materials should be as shown in Fig. 12.10 and Table 12.2. The total base-course, as shown, will be approximately 80 cm.

Where the subgrade soil condition is **very poor** and has a CBR of approximately 5 per cent, the construction and use of the materials should be as shown in Fig. 12.11 and Table 12.3. The total base-course, as shown, will be approximately 100 cm.

Bedding layer

Between the concrete paver and the base-course, a layer of bedding sand is required. The bedding should be constructed in a thin layer

Table 12.1. Construction layer and materials for CBR ≈ 25 per cent or more

Construction layer and type of materials	Thickness in cm
Pavement: inter-locking pavers	8–10
Pavement: rectangular pavers	10–12
Bedding layer (crushed rock: 0–8 mm)	3
1. Base (upper) (crushed rock: 0–30 mm)	5
2. Base (lower) (crushed rock: 0–60 mm)	10
Subbase (crushed rock: 0–150 mm)	30
Other	–
Subgrade	Existing

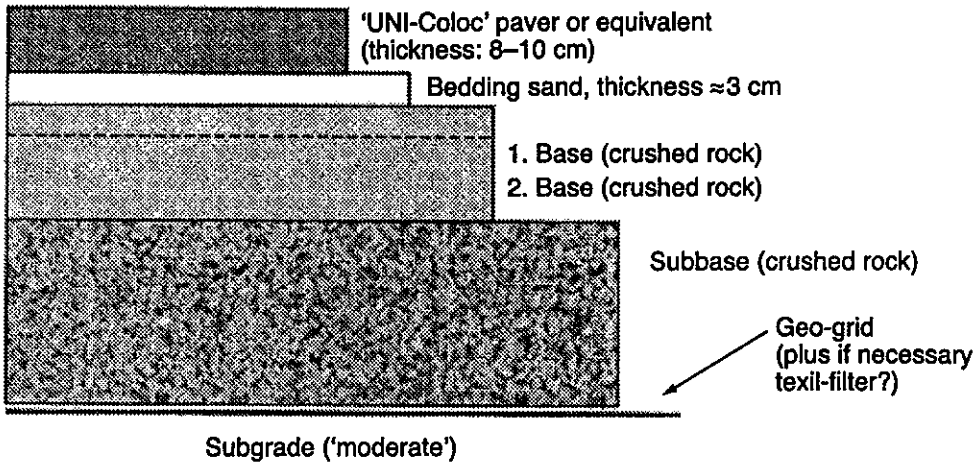


Fig. 12.10. Typical construction for subgrade of CBR ≈ 10 per cent

to give maximum stability. The following guidelines and recommendations should generally be followed for the construction of the bedding layer:

(a) Material to be crushed rock grading 0–8 mm (maximum 0–11 mm).
(b) Compressed (mill) layer with average thickness of approximately 30 mm. Local deviation for thickness, maximum ±10 mm (ref. theoretical thickness 30 mm).
(c) Material to be moistened during finishing of the bedding layer.
(d) Geometric tolerance for top bedding layer; same as for top pavement.
(e) Suitable materials for good drainage.
(f) Materials should meet the quality requirements in the national standards.

Table 12.2. Construction layer and materials for CBR ≈ 10 per cent

Construction layer and type of materials	Thickness in cm
Pavement: inter-locking pavers	8–10
Pavement: rectangular pavers	10–12
Bedding layer (crushed rock: 0–8 mm)	3
1. Base (upper) (crushed rock: 0–30 mm)	5
2. Base (lower) (crushed rock: 0–60 mm)	15
Subbase (crushed rock: 0–150 mm)	60
Geo-grid	(e.g. 'Tensar SSLA30')
Subgrade	Existing

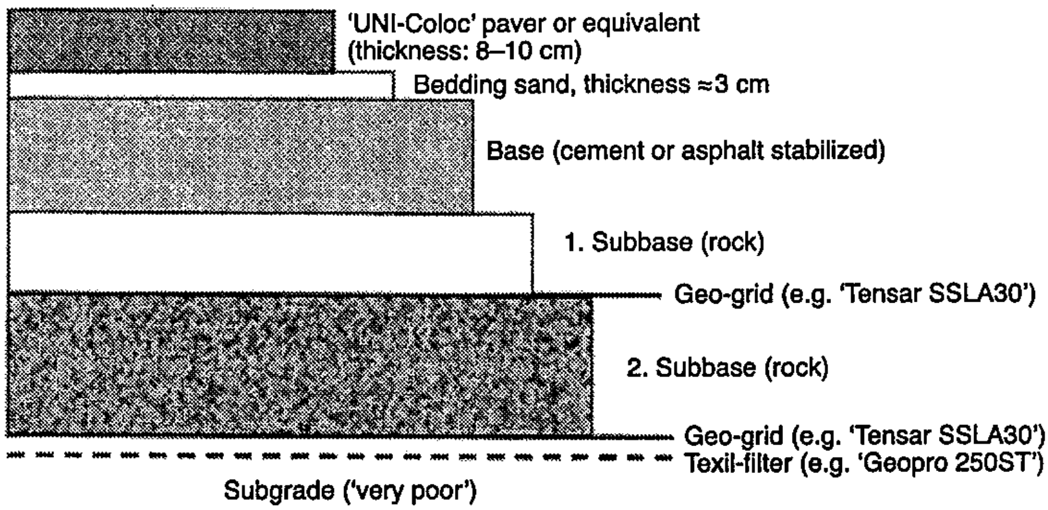


Fig. 12.11. Typical construction for subgrade of CBR ≈ 5 per cent

For all types of pavements, there could be a problem with settlement close to solid structures, like foundations and concrete slabs, drains, etc., due to difficulties in compacting the base-course. The problem usually appears after a long period of use. To maintain a proper cross fall, the pavement should be constructed with an increased elevation close to the structures. The increase in the thickness of bedding layer could be approximately +10 mm gradually over 1–2 m.

Types of block pavers

In foe areas, exposed to heavy loads, it is important to choose a block paver of suitable and robust design and geometry.

Table 12.3. Construction layer and materials for CBR ≈ 5 per cent

Construction layer and type of materials	Thickness in cm
Pavement: inter-locking pavers	8–10
Pavement: rectangular pavers	10–12
Bedding layer (crushed rock: 0–8 mm)	3
Base (cement or asphalt stabilized)	20
1. Subbase (upper) (rock: 0–80 mm)	20
Geo-grid	(e.g. 'Tensar SSLA30')
2. Subbase (lower) (rock: 0–150/200 mm)	60
Geo-grid	(e.g. 'Tensar SSLA30')
Texil-filter	(e.g. 'Geopro 250ST')
Subgrade	Existing

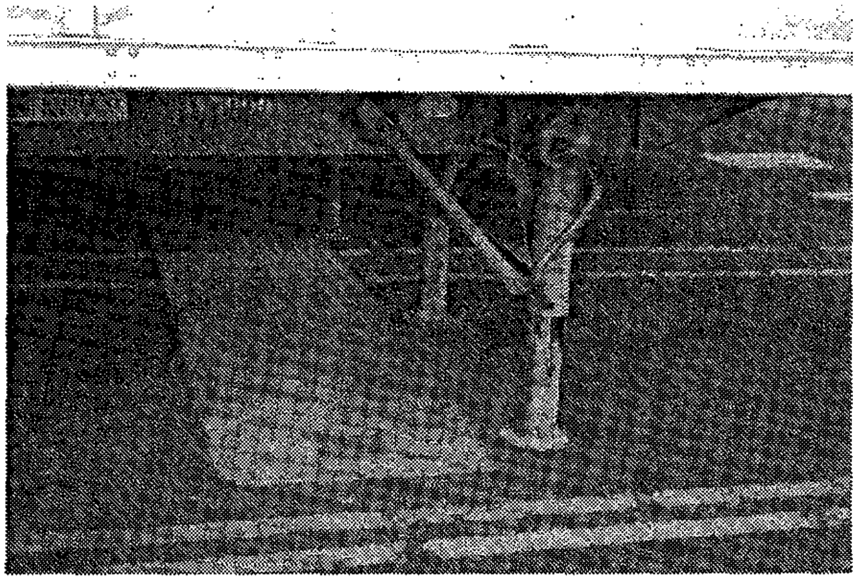


Fig. 12.12. Damages in an asphalt paver

In the international literature, pavers are classified in the following three categories:

(a) **Category A** comprises dentated pavers which key into other pavers on all vertical faces like 'UNI-Coloc' ('UNI-Anchorlock' in the USA) or equivalent.
(b) **Category B** comprises dentated pavers which key into other pavers on only two faces like 'SF-Paver' or equivalent.
(c) **Category C** comprises non-dentated pavers which do not key into other pavers like hexagonal and rectangular pavers or similar shapes.

Experiences have shown that interlocking concrete pavers have higher load capacity than asphalt, particularly on warm days, as shown in Fig. 12.12, where supports for the container or the container corners can penetrate the asphalt. Based on test results concrete paver layers with a thickness of 80 mm have a relatively high modulus of elasticity of $E \approx 6000–7000$ mPa (Category A), compared to an 80 mm-thick asphalt layer of $E \approx 3000–4000$ mPa.

Generally the concrete thickness of the concrete block pavers used in harbours worldwide are between 8 and 12 cm. Some of the largest harbours, for example in the Netherlands, are constructed with ordinary rectangular block pavers up to 12 cm thick. The harbours in Norway are mainly constructed with an 8 cm thickness of interlocking concrete pavers. Therefore with Category A it is possible to reduce the thickness of the concrete pavers, due to better performances and less movement and rotation than for categories B and C. The following guidelines and recommendations should generally be followed for the selection of pavers:

(a) Quality of concrete pavers should meet the minimum requirements in the national standard.
(b) Type of paver. It is recommended, for example, that container terminals use Category A. If rectangular pavers are chosen, it is recommended that the thickness of the paver be increased and laid in a herringbone-pattern to increase the pavements' stability.
(c) Thickness of paver should be 80 mm (100 mm when extreme loads) for Category A and a thickness of 100–120 mm for Category B or C.

Laying pattern

There are many different laying patterns for various kinds of concrete block pavers for heavy duty areas such as container terminal areas. Some types of paver may require a specific laying pattern. Rectangular pavers in stretcher pattern in Category C will increase the pavement performance if laid in a herringbone pattern, but will never achieve the same level as the best designed paver in Category A.

The best result from an area pavement is achieved by block pavers of Category A with an interlocking pattern. Comprehensive neutral tests and long experience prove that these kinds of block pavers contain the best characteristics. They are also designed for effective machine laying.

The pavements' performance will be improved if the direction of the pattern system is twisted 45° to the main traffic direction. To get the best effect of interlocking, the joints between block pavers need to be less than 5 mm. The following guidelines and recommendations should generally be followed for the laying of the paver:

(a) Use pavers in an interlocking system as for Category A.
(b) If the pavers are not designed for pattern, use a herringbone pattern if possible.
(c) Pattern twisted 45° to the traffic direction.
(d) Joint width should be an average of 2–3 mm and should be less than 5 mm.

Performance of a pavement

Block pavers for areas larger than 2000–3000 m^2 are normally laid by machine as shown in Fig. 12.13. One machine can carry out up to

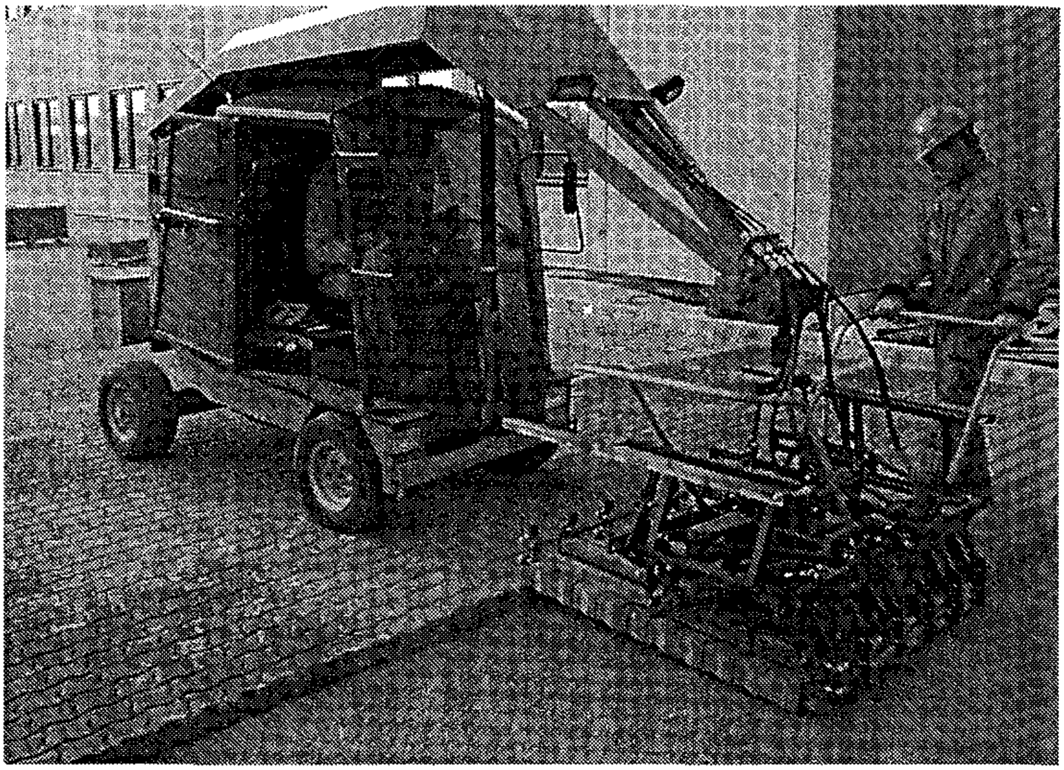


Fig. 12.13. Concrete pavers laid by machine

1000 m^2 of pavement/10 h. The following guidelines and recommendations should generally be followed:

(a) Use interlocking-pavers of Category A with high concrete quality.
(b) Use a well-recognized contractor with relevant references.
(c) Before the start of paving, check the level and evenness of the bedding layer.
(d) The paving pattern should follow straight lines with average jointing width of 2–3 mm.
(e) Use dry jointing sand and a vibrator to fill the joints completely.
(f) Survey and document the completed pavement level and elevations.

Further reading

British Standard BS 6349 (1988) *Maritime Structures. Part 2: Design of Quay Walls, Jetties and Dolphins*, London: BSI.

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European Standard; CEN-1338: Concrete Paving Blocks. Requirements and Test Methods, Central Secretariat, Brussels (to be published).

NASA- & SAE-Test No. 921036 & 922013 (1992) SAE International Mobility Land Sea Air and Space, Warrendale, PA, USA.

Shackel, B. (1990) *Design and Construction of Interlocking Concrete Block Pavements*, Elsevier Science Publishers Ltd., London.

13

Container terminals

13.1 Site location

Container terminal planning and evaluation can be a very complex task. The designer must make the most of the available local resources to meet the required level of productivity, while trying to reach a balance between the needs of the port authorities, port operators, stevedoring companies and container shipping lines. The capacity of a port is commonly expressed by the amount of cargo throughput and the efficiency by its ability to handle cargo or containers with a minimum of costs.

A survey must be carried out to identify existing and potential sites so as to meet the activity requirements of the port. A port plan must also indicate areas earmarked for future port expansion and establish guidelines for such development. The goal for all port development should be the possibility of working day and night, 24 hours around the clock, 7 days a week and 365 days a year.

Port improvements frequently enable the shipping in the port to turnaround more quickly, either through reduced waiting, or through more efficient cargo-handling operations, that result in a reduced berth service time. The quicker handling of cargo, whether transfers from ship to berth, from berth to storage, or to and from land transport systems usually results from improved mechanization of the berth facilities.

From the moment a container arrives at the port, either at the port gate or at the berth side, it should be logged into a computerized system that can track the container through each stage of its transit through the container terminal. By this way the customers can know the status of their container at any time.

Sometimes quite significant improvements can be made by reorganization and improved management systems, for instance by establishing one terminal operator. Therefore improvements to the port facilities and organization, together with port layout improvements, will, in most cases, result in more efficient handling and storage of the cargo.

13.2 Existing areas

The capacity of existing berth facilities and port areas has to be assessed. New loading and unloading methods usually have the result that the bottleneck for port efficiency is no longer a lack of berth capacity but a lack of areas and installations ashore. On many older port terminals, the area close behind the berth front contains too many sheds and buildings, so that there are hardly any open areas for handling and storage of containers and large units of cargo. Therefore one must evaluate whether relocations within the existing port area can increase port output to a modern container port as shown in Fig. 13.1. The figure shows a possible solution from an area point of view for an old general cargo jetty, which has been converted into a modern container terminal.

When assessing the effective output of existing port areas, the following points must be considered:

(a) technical level
(b) operational level
(c) storage capacities
(d) ownerships

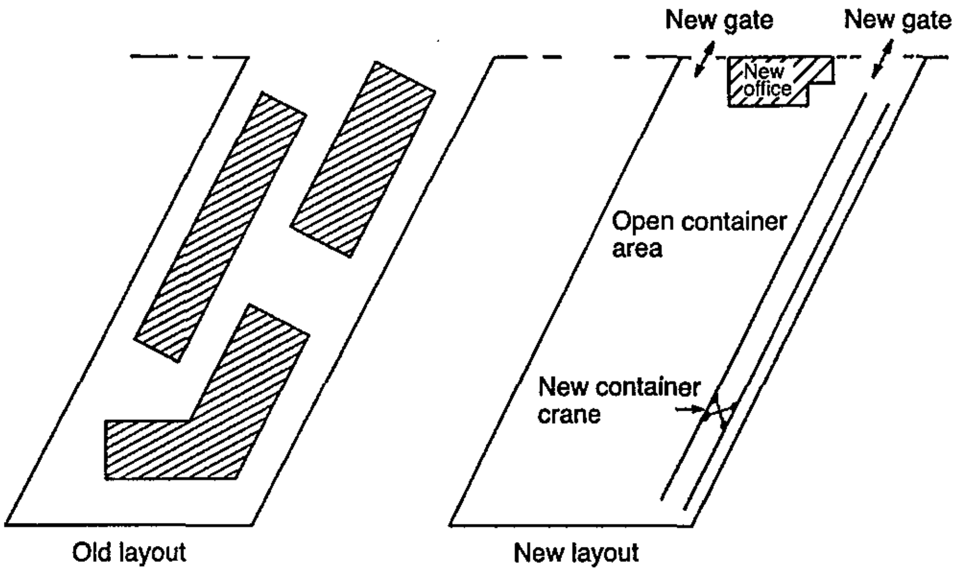

Fig. 13.1. Relocation of an existing port area to a container port

(e) possibility of relocations of existing facilities
(f) environmental considerations.

Through improved extensions of land area behind the berth, the port capacity could be increased. This means that the capacity of a port today is more dependent on efficient management and available space on land than on the length of the berth front itself.

13.3 Potential areas

In many ports it can prove difficult to find suitable areas for port expansion adjacent to the existing port area. Very often the nearby surroundings of existing ports are so crowded and restricted, owing to town development, that direct expansion of the existing port facilities is more or less impossible. Therefore, one must survey the stretches of the coast where port development may be possible and include them in the overall port plan. This survey must take into account the following factors:

(a) availability of sufficient area
(b) possibility of future extension
(c) availability of hinterland connections
(d) accessibility and distance from sea
(e) nature of subsoil and risk of settlements or geotechnical problems
(f) shelter from waves/wind/current
(g) earthquake danger
(h) environmental assessment.

In particular, the transhipment operations of containers and ro/ro cargo are very sensitive to ship movements due to, for example, wind and waves, and this can lead to considerable downtime. The location of a port or terminal should therefore be chosen with the utmost care.

13.4 Container ships

The aim for a terminal is to accommodate the largest container ship without it waiting to berth. For the largest container ship in service nowadays, the berth facilities should be designed to accommodate the ship at any tide level, with a waterfront length of the berth basin of approximately of 400 m, a ship beam of minimum 45 m and a draft of 15 m.

The sizes of the container ships have increased since *Sea-Land* introduced the container concept in 1956. Nowadays the following approximate grouping of container ships is used.

Fig. 13.2. 1st generation container ship

Feeder ships have the task of collecting containers from smaller ports and carrying them to the main container port. The feeder ships usually vary in size from approximately 50 TEUs (20-ft equivalent units) up to 300 TEUs.

Panamax-size container ships with a width up to approximately 32 m:

(a) 1st generation with capacity up to about 1000 TEU, as shown in Fig. 13.2
(b) 2nd generation with capacity up to about 1600 TEU
(c) 3rd generation with capacity up to about 3000 TEU.

Post-Panamax-size container ships:

(a) 4th generation with capacity up to about 4250 TEU
(b) 5th generation with capacity up to about 5000 TEU
(c) 6th generation with capacity up to about 6000 TEU, as shown in Fig. 13.3.

Post-Panamax-Plus-size (PPP) container ships:

(a) 7th generation with capacity over 7000 TEU.

ULCSs (ultra large container ships):

(a) These container ships have an overall capacity of 12 500 TEU or more, maximum length of approximately 380–400 m, a ship beam of 60 m and a maximum design draft of 14.5 m. The design speed is between 23–25 knots.

There are, nowadays, container ships in service which are capable of carrying approximately 8000 TEU, and, on the drawing board, there are

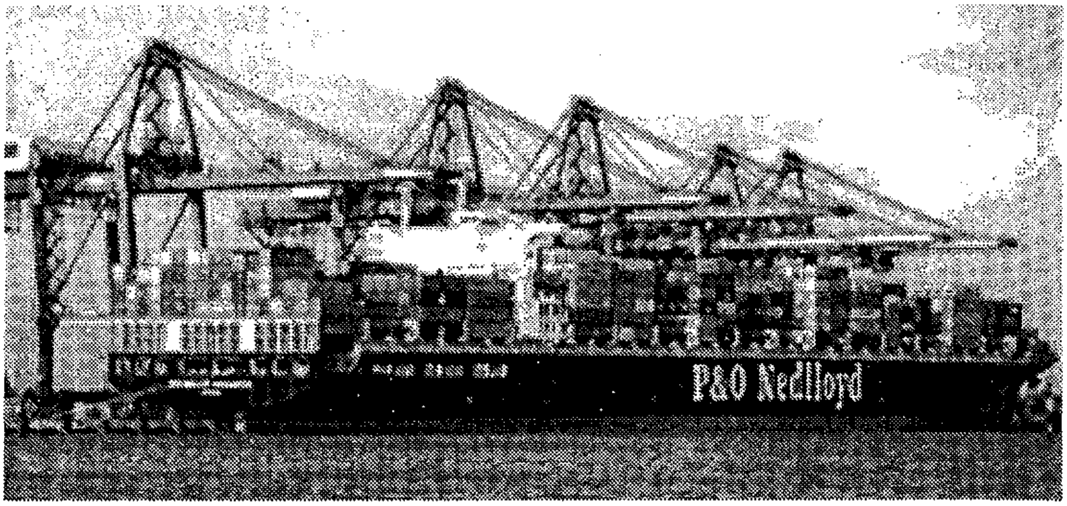

Fig. 13.3. Post-Panamax container ship

ultra-large container ships with a carrying capacity of approximately 15 000 TEU. These ships have a cell width of 22 containers on deck and 20 containers below, with a length of approximately 400 m, a beam of approximately 60 m and a maximum design draft of 14.5 m.

13.5 Terminal areas

In the evaluation of new potential port areas it is useful to divide the new potential land area behind a new berth line into an apron, a primary and a secondary yard, and a storage area. The length of the land area or berth will depend on the type of ship and/or cargo to be expected. For medium container ships (2nd generation container ships) and multi-purpose ships, a length of about 200 m will be sufficient for one berth.

The total terminal area is usually divided into the following:

(a) The apron or the area just behind the berth front.
(b) The primary yard area or container storage area.
(c) The secondary yard area, which includes the entrance facility, parking, office buildings, customs facilities, container freight station with an area for stuffing and stripping, empty container storage, container maintenance and repair area, etc.

13.5.1 Apron

The width of the apron will vary between about 15–50 m depending on the loading and unloading equipment, trucks, cranes, etc. The

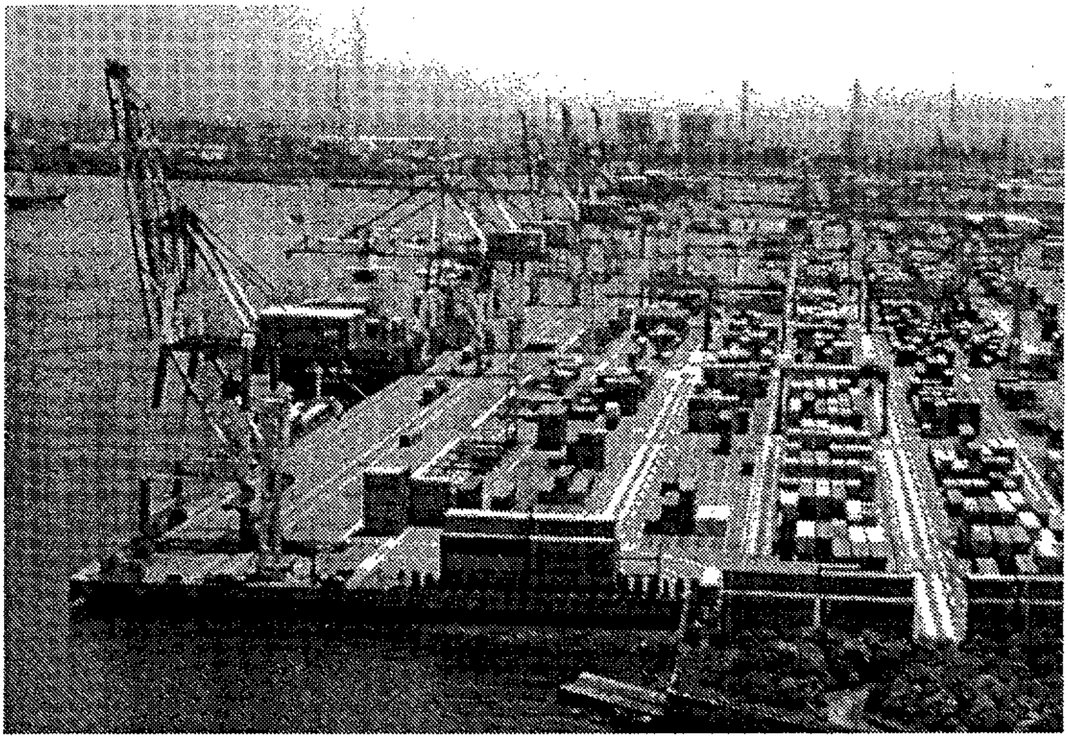

Fig. 13.4. Layout of a modern container terminal

dimensions of the various sections of the width of the apron for a berth with crane will be:

(a) The distance from the berth line to waterside crane rail should not be less than 2.5 m and contain the crane power trench, bollards, the gangway and other ship utilities.
(b) The distance between the crane rails varies from about 10 m (general cargo crane) to a maximum of 35 m (container crane).
(c) The traffic area or road behind the landside crane rail and the boundary between the apron and the primary yard can vary in width from 5 m to 15 m.

13.5.2 Yard area

The yard or area behind the apron may be divided into a primary yard and a secondary yard with entrance, parking, office building, custom facilities, etc. The primary yard or the storage area is the area immediately adjacent to the apron and is used primarily for storing inbound and outbound cargo. The secondary yard is the area for storing empty containers, equipment, etc.

The yard area for a modern multi-purpose and container terminal, like the port shown in Fig. 13.4, should have a depth of at least

300 m behind the apron. Preferably the area should be up to about 400 m for a multi-purpose terminal and up to about 700 m for a modern container terminal.

Therefore, the land requirements are related to the storage density and the time the cargo stays in the port. Where a substantial proportion of the cargo is handled by ro/ro methods, the back-up areas can be much larger than for cargo handled by lo/lo methods. As a rule of thumb the area required for a multi-purpose terminal will vary between about 5–15 ha/berth, and for a container terminal about 10–100 ha/berth depending on the generation of the container ship. These figures include areas for offices, sheds, workshops, roads, etc.

Generally the **total yard area** can be divided into:

$$A_T = A_{PY} + A_{CFS} + A_{EC} + A_{ROP}$$

where

A_{PY} = the primary yard area or container stacking area. The area is approximately between 50–75 per cent of the total area

A_{CFS} = the container freight station (CFS) with area for stuffing and stripping, etc. The area is approximately between 15–30 per cent of the total area

A_{EC} = the area for empty containers, container maintenance and repair area, etc. The area is approximately between 10–20 per cent of the total area

A_{ROP} = the area for entrance facility, office buildings, customs facilities, parking, etc. The area is approximately between 5–15 per cent of the total area.

In the evaluation of the total yard area, the area for entrance, custom facilities, etc. will be affected by the proposal by the International Maritime Organization (IMO) to improve the security in the port.

When there is little knowledge of the expected cargo in the future, the storage area should have an additional area of between 25 and 40 per cent as reserve capacity. To provide less than a 25 per cent reserve would be unwise under any circumstances.

Roads may in a ro/ro container terminal occupy up to about 50–60 per cent of the total area, and in a lo/lo container terminal occupy up to about 40–50 per cent of the total area. Records from major European container ports show that a minimum 30 per cent of the total port area is used for roads, port services, parking, rail rods, etc.

Figure 13.4 shows a general layout of a total yard area of a terminal area that has most of the container terminal activities.

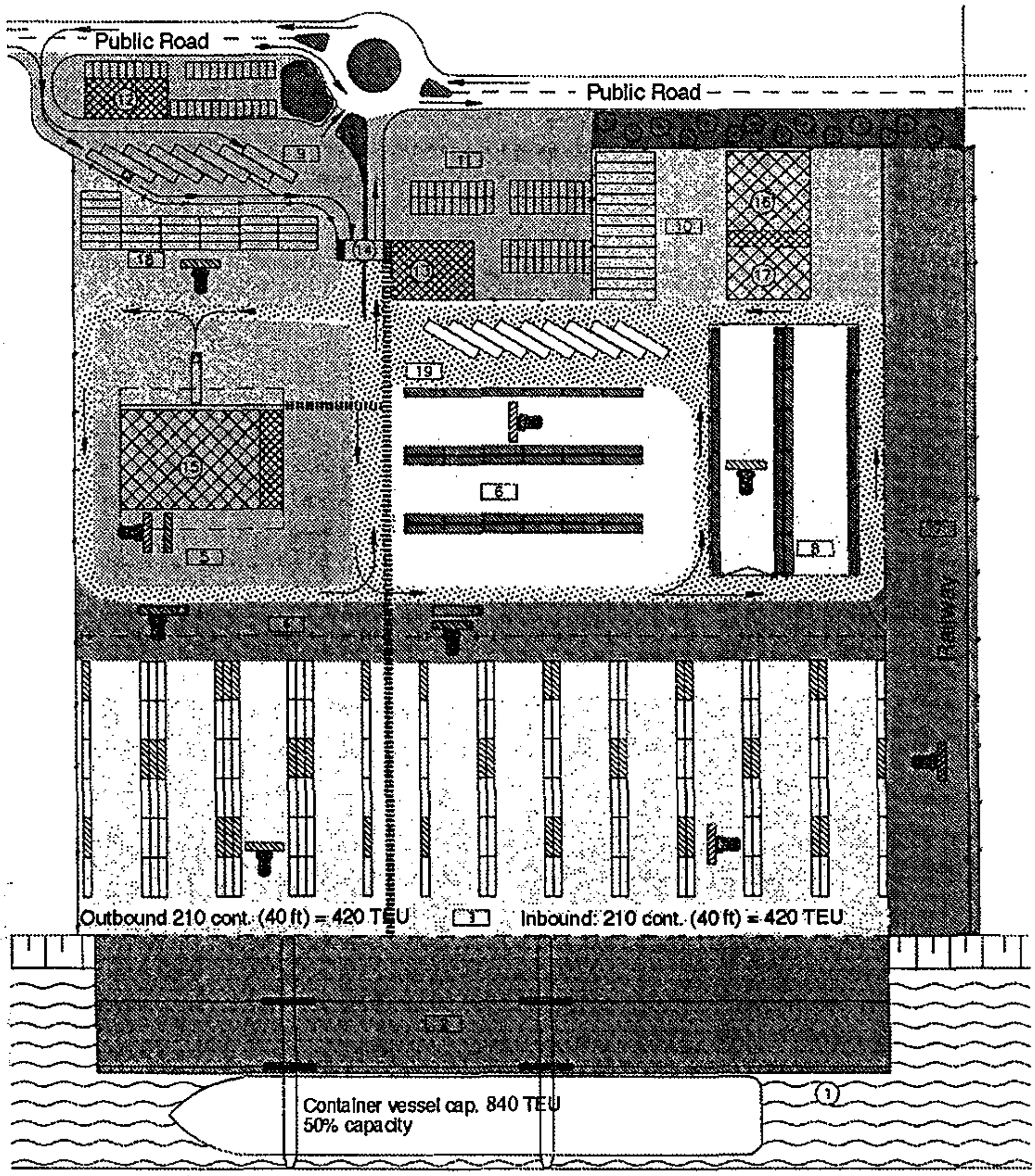

Fig. 13.5. A general prototype of container terminal yard area

Where the different notes on Fig. 13.5 are:

1	= berth
2	= apron
3	= stacking and storage area
4, 7	= internal road and terminal transfer system
5, 15	= stuffing and stripping area and shed
6	= reefers area
8	= area for dangerous cargo, etc.
9	= terminal entrance

10, 16, 17 = service, repair, workshop equipment and workshop container area
11 = terminal landside
12 = load identification area
13 = office, canteen and convenience
14 = in and out checking
18 = depot for empty containers
19 = parking area for vehicles.

13.6 Ship to shore crane

Since the first specially designed container crane was completed in 1959, the ship to shore (STS) crane has been a fantastic development in the design of the container cranes. Figure 13.6 shows an example of the development of a container terminal.

The 3rd generation container ship can stow container boxes up to 13 rows across, while the 6th generation post-Panamax can stow 18 rows across and in the future one must expect container ships that will be 22 containers wide. This increase in size must be met with sufficient crane capacity, because as the ships get larger their time becomes more precious and time spent at the berth is unproductive time. Nowadays the larger ships use up to four or five container cranes simultaneously.

The traditional STS crane system can be inadequate to properly serve the existing and, especially, future generations of PPP container ships efficiently. To increase the possibility of serving the ships faster, one can either:

(a) generally increase the crane efficiency
(b) increase the crane rate by lifting two loaded container simultaneously
(c) introduce a dock system where one can load and unload the ship from both sides.

The large PPP ships are very expensive to run, so any delay in loading or unloading can be very costly in reducing the economic benefits that otherwise result from running a larger container ship. For example if a 6500 TEU ship were to load and unload 80 per cent of its capacity with three cranes of 30 lifts/h, 21 h/day, the total ship call would last more than 3 days.

Owing to the size of the PPP ships and their enormous costs/h they demand an optimal turn-around time of approximately 24 h. This

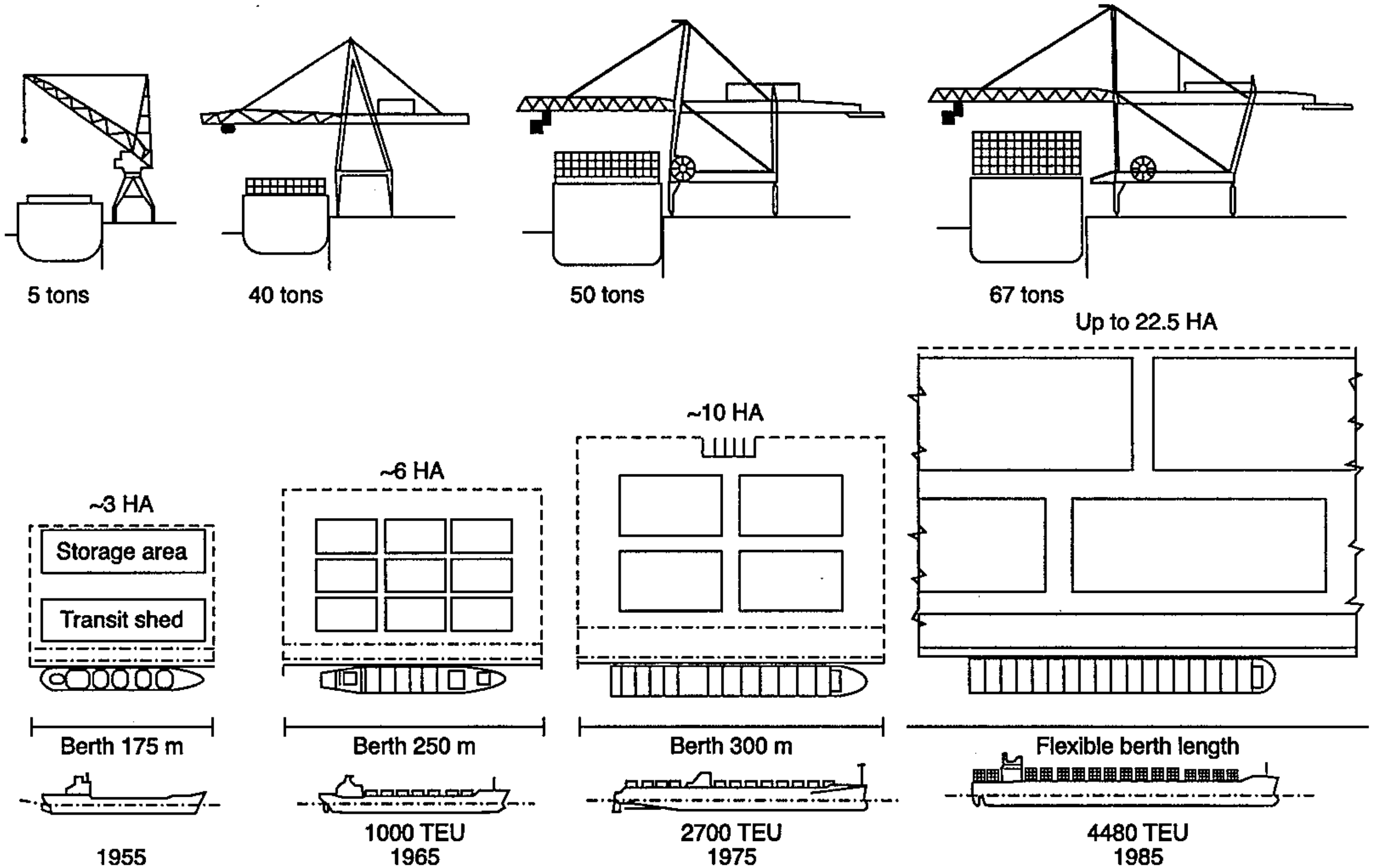

Fig. 13.6. A general development of container terminals

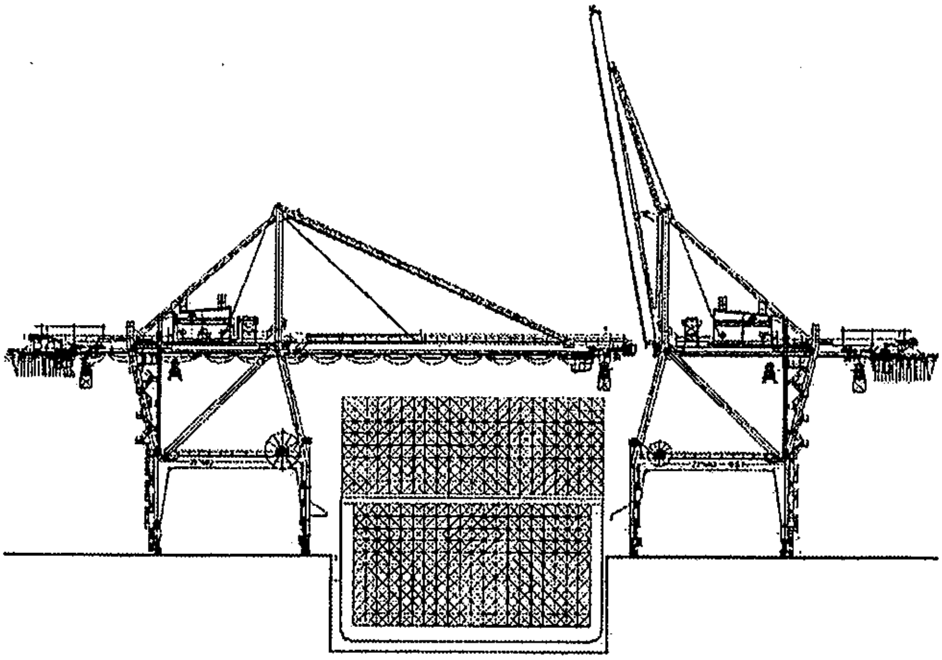

Fig. 13.7. Cross-section of the dock system

means that the time for unloading and loading will be approximately 22 h and 2 h for sailing and manoeuvring in the port. Therefore, to keep or improve the productivity/berth h, the terminal operator must use up to four or five ship to shore gantry cranes on the larger container ships.

For the traditional container berth system, the average required berth production rate for an 8000 TEU ship would, with a load factor of 0.85 and a TEU factor of 1.5, be approximately 250 moves/h or 5 ship to shore gantry cranes each with an effective capacity of approximately 50 moves/h.

The companies JWD and Liftech, in Holland, have evaluated a very interesting dock system for loading and unloading very large container ships, as generally shown in Figs 13.7 and 13.8, with loading and unloading from both sides. By this system one can increase the number of cranes working against the container ship to twice as many and therefore reduce the stay in the port to approximately half the time. The ship in a dock system can use the existing berth crane technology and operating systems. The dock should have width and depth large enough to accommodate the future container ships. The size of the dock with a two side-handling system will approximately have a width up to 70 m and a length of 380–400 m.

The design and layout of the surrounding container terminal around the dock could be more difficult since the terminal yard would need to

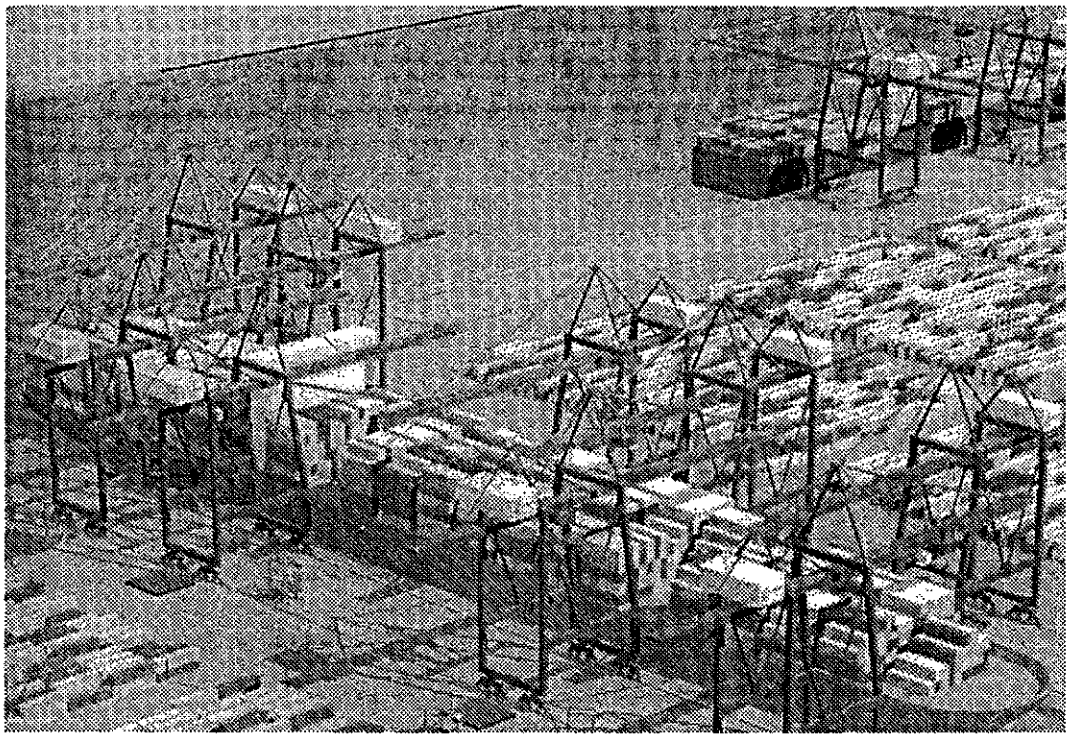

Fig. 13.8. A layout view of the dock container system

wrap around the dock. Economically, the dock system would need construction of berth on both sides of the ship and on the end of the dock. The dock with a two-side loading and unloading system can be summarized as follow:

(a) The berthing and unberthing operation to and from the dock could take longer time.
(b) Per container ship one needs twice the berth length and width of dock.
(c) The orientation of the dock, due to the stacking area, could require more space or terminal area than one-side berthing.
(d) The berth may not be suitable for all ship sizes.
(e) The container cranes cannot be transferred from one berth to another.

The conclusion is that the traditional berth with one-side handling could be preferred, but due to the increase in container ship size one has to evaluate these advantages and disadvantages very carefully.

Some of the world's major container ports have an average of between 1.6–1.8 TEUs per crane lift, meaning that 60–80 per cent of the lifts are 40 ft containers, but these have to be evaluated in each port's case.

The **crane capacity/h** for handling containers can vary between 10 and, at the extreme, 70 containers, with an average capacity of about 25 containers/h per crane. As for guidance only, the following can be used:

(a) Rail-mounted harbour cranes can handle from ship to shore about 15 containers/h.
(b) Mobile container cranes about 15–25/h.
(c) STS gantry cranes about 30–40/h. For ship to shore gantry cranes with a secondary trolley system 40–70/h.

In a feasibility study one shall take into consideration a loss of output for opening and closing the total crane operation of about 10 per cent of the basic output rate. The working crane time to the time the ship stays at the berth is about 80 per cent.

The modern ship to shore gantry cranes have, with single container duty cycles, an average of 90–120 s, or about a theoretical 30–40 container lifts/h depending on a properly-fed delivery service to and from the container stacking yard. The truck-based terminals have productivity generally limited to about 28–35 lifts/h, while the straddle-carrier-based terminals can support the highest ship to shore crane productivity, with good management systems, of up to 40 lifts/h.

For the largest port, up to six cranes can work simultaneously on the largest PPP container ship on the same side for a short period but, practically, one should assume that only four cranes can work simultaneously. With the dock system of cranes from both sides, the number of cranes working simultaneously can be high since the standard crane booms are narrower than their supporting frames, and the cranes on the opposite sides of the container ship could theoretically be nested with opposite booms on alternative hatches.

An interesting survey about the average handling rates actually being achieved around the world showed that the crane performances are below the manufacturers' stated moves/h capabilities of modern cranes. The percentage analysis of average handling speeds for container cranes based on a survey of 671 cranes worldwide were:

(a) up to 20 moves/h	12%
(b) 21–25 moves/h	39%
(c) 26–30 moves/h	33%
(d) 31–35 moves/h	14%
(e) over 35 moves/h	1%

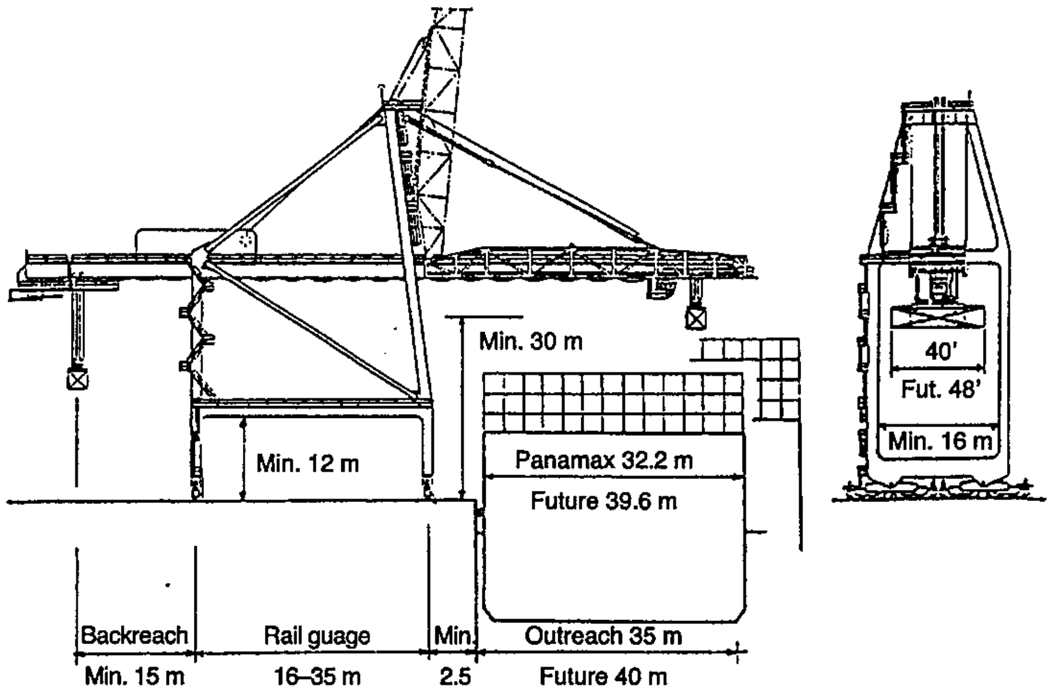

Fig. 13.9. A general cross-section of a container berth

The most commonly used specialized ship to shore container gantry cranes, as shown in principle in Figs 13.9 and 13.10, feature the following items:

(a) Minimum height between the lower part of the spreader and the berth level 30 m. Generally the gantry crane must be able to

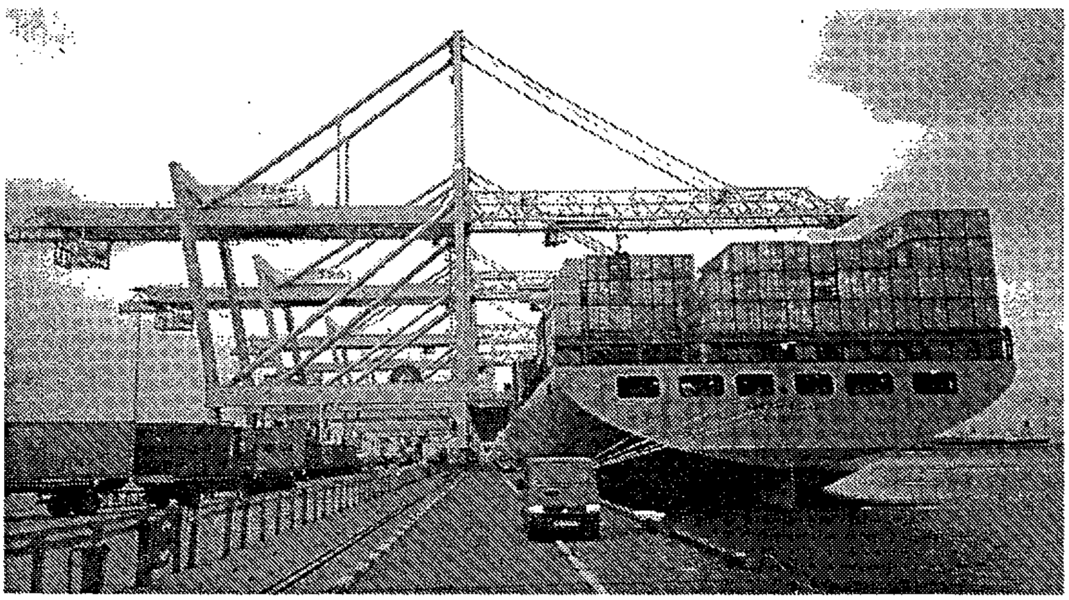

Fig. 13.10. A 6th generation container ship at the berth

stack 5 high on deck on a large ship and must be able to have access to every individual container on the ship.

(b) For larger container vessels the distance from the fender front to the front crane rail should be 7.5 m due to the shape of the bow and the berthing angle of the larger container ships during the berthing operation.
(c) Minimum outreach, measured from the fender front of the face of the berth, should be 35 m, with maximum outreach of about 45.2 m, and minimum back reach of 15 m.
(d) The outreach must reach 13 containers wide for Panamax ship and up to 22 wide for PPP.
(e) Rail gauge between 16 m and 35 m. A rail gauge of about 35 m will allow six truck lanes between the crane legs. The distance between the rail gauges is usually not a crane stability problem, but is more determined by the operating system between the apron area and the stacking area.
(f) A clearance of at least 16 m is provided between the legs in order to leave room for containers and cargo hatches.
(g) The maximum width of the gantry crane (buffer to buffer) should not be more than 27.5 m.
(h) For the PPP gantry crane, the weight of the crane, the load of the max container, wind and dynamic effects and with four sets of standard support of 8 wheels, the weight on each wheel would be approximately 65 tons or 70 tons/lin m crane rail.
(i) At least 400 kN lifting capacity under the spreader. For twinlift spreaders up to 660 kN lifting capacity.
(j) Lifting velocity about 3 m/s with empty spreader, adjustable to 1.50 m/s with rated load.
(k) Programmable operation control and failure detection system.
(l) Alarm for excessive wind speed and emergency shut off.

If the average crane capacity is increased to about 40–50 or more containers/h, the total terminal capacity, i.e. the area needed, storage and delivery capacity, etc., has to be increased tremendously.

For this reason the main benefits of a port improvement or development could be savings in ship waiting time and service time. Large costly ships require efficient ports which minimize the ship turnaround, because improvements which reduce the waiting time to call at the port and the time spent at berth, etc., can save the ship owner large sums in operating expenses. Such savings will be reflected in the freight rates.

13.7 Container handling systems

The most commonly used container handling systems nowadays for stacking the containers at the container stacking areas are:

(a) The fork-lift truck and reach-stacker system.
(b) The straddle-carrier system.
(c) The rubber-tyre gantry (RTG) and/or rail mounted gantry (RMG) system.
(d) A mixture of the above systems.

Generally all terminals should have a buffer storage area in front of the storage or stacking area. One should also take into account that all terminal equipment will have a reduction in capacity due to service and repair of between approximately 5–30 per cent.

13.7.1 Stack height

The stack height will affect both the total storage capacity of the stack and the accessibility to the individual boxes within the stack. With the need for storage and the limited space available, the tendency will be to adopt a high stack in order to maximize this storage capacity.

However, increasing the height of the stack also reduces the accessibility of individual containers within the stack as individual containers become buried deeper. This will increase the amount of 'digging' by terminal equipment to retrieve containers and hence to an overall reduction in terminal efficiency or an increase in equipment. Therefore a limit to the stack height occurs when the need to allow enough spaces to place the other containers during digging is imposed.

Digging for export containers can be reduced by careful planning of container placing within the stack, in line with shipping schedules. However, it is difficult to reduce the digging for the transfer of import containers, as the collecting lorries are likely to arrive at random, making planning impossible. An optimum balance between these factors must therefore be found for the stack height.

13.7.2 Fork-lift truck and reach-stacker system

The STS gantry crane places the containers on a terminal tractor system and move the container to the stacking area where the container is stacked by a fork-lift or a reach-stacker system as shown in Figs 13.11, 13.12 and 13.13.

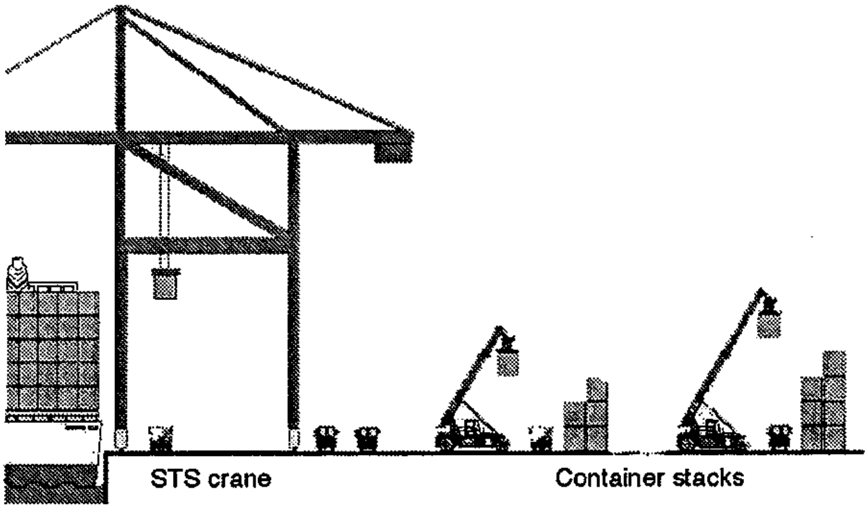

Fig. 13.11. Stacking by reach stackers and by using terminal tractors between the STS gantry crane and the stacking area

Heavy fork-lift trucks with top loaders have traditionally been used for container handling. Nowadays, more operators use the reach-stacker system because of the higher productivity and higher stacking density. The system with fork-lift and reach stackers can be the most economical and commend for small terminals handling up to

Fig. 13.12. A terminal tractor

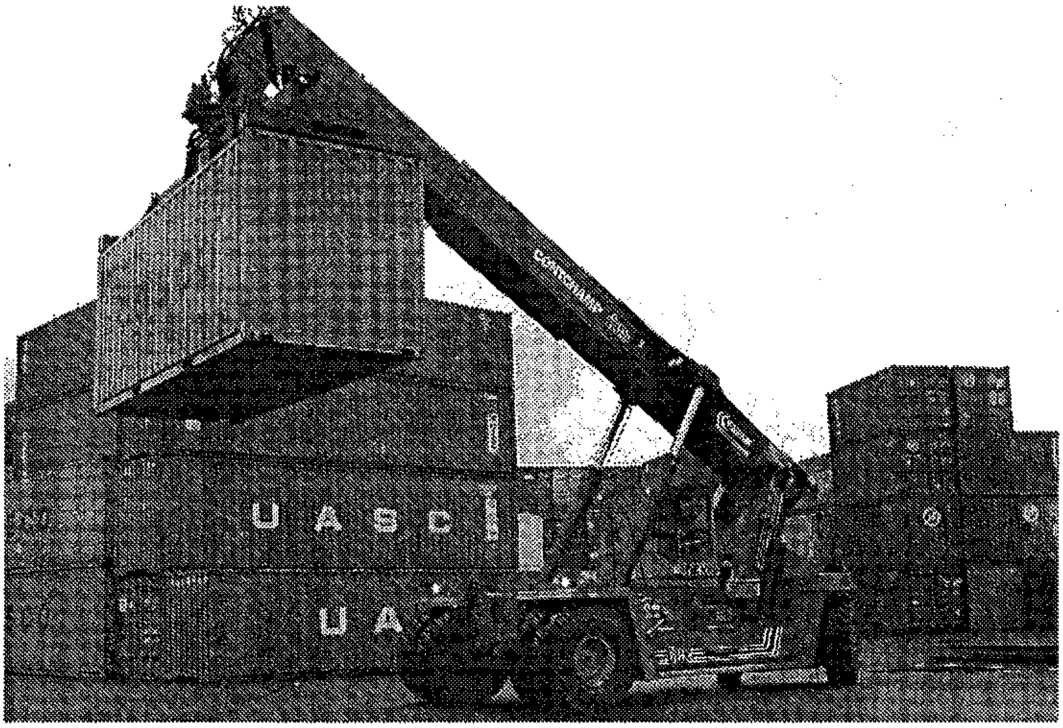

Fig. 13.13. A reach stacker

approximately 60 000 to 80 000 TEUs per year and where the size of the terminal area is not restricted, while the reach-stacker system can be used economically for container handling at terminals with capacities up to approximately 200 000 to 300 000 TEUs. The reach-stacker system can stack containers 4 deep and up to 6 containers high, but normally the stacking is 2 deep and 3–4 high to avoid too much reshuffling of the stack.

Generally:

(a) 3 to 5 terminal tractors and 2 reach stackers per STS gantry crane. The numbers of terminal tractors are dependent on the distance between the berth and the stacking area.
(b) Low-storage capacity with about 500 TEUs/h, stacking the containers approximately 4 high.
(c) Medium STS crane productivity and no buffer zone under the STS crane.
(d) High labour, but low capital and operating costs.
(e) Low control, trucks allowed to stacking area.

13.7.3 Straddle-carrier system

The ship to shore gantry crane places the containers on the apron where the straddle carrier moves the container to the stacking area, and the

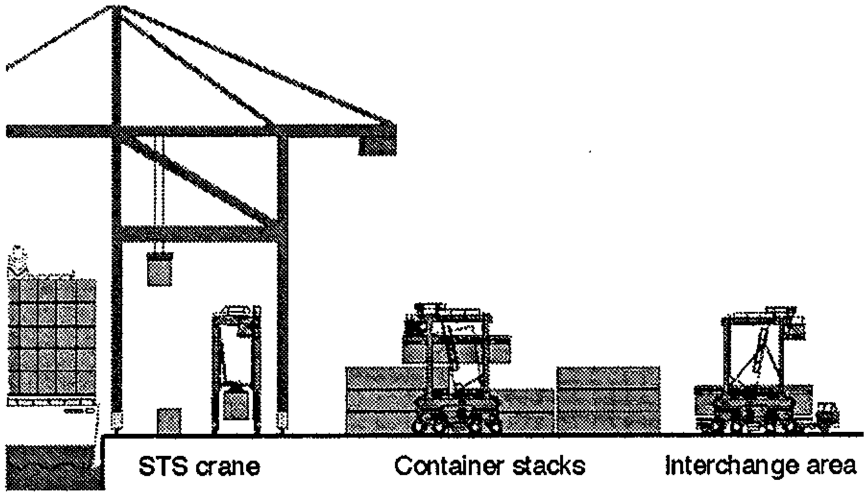

Fig. 13.14. Stacking by straddle carriers and by also using straddle carriers between the STS gantry crane and the stacking area

container is stacked by straddle carrier as shown in Figs 13.14 and 13.15. The straddle-carrier system is an independent system, and does all the different handling operations from the STS crane to the stacking of the containers.

The straddle-carrier system is well suited for ports that have small terminal areas, and it is later easy to alter the layout of the terminal. There is normally no need to reinforce the terminal pavement because the straddle-carrier's wheel loads are much lower than the reach-stacker's wheel loads.

The main benefits of the straddle carrier in relation to the reach-stacker system are: savings in labour costs, more ground slots in the same area, and easier and direct access to the containers resulting in improved selectivity and less unproductive moves.

Usually the straddle-carrier system stacks the containers two or three high. The straddle-carrier system is usually the fastest system for terminals handling between 100 000 and up to about 3 000 000 TEUs per year.

Generally:

(a) 3–5 straddle carriers per STS crane depending on the distance between the berth and the stacking area.
(b) Approximately 10 moves/h per straddle carrier.
(c) Medium stacking density, with about 750 TEUs/ha with stacking of containers 3 high.
(d) High STS crane productivity and buffer zone under the STS crane.
(e) Low labour, but high capital and operating costs.
(f) High control, trucks not allowed to stacking area.

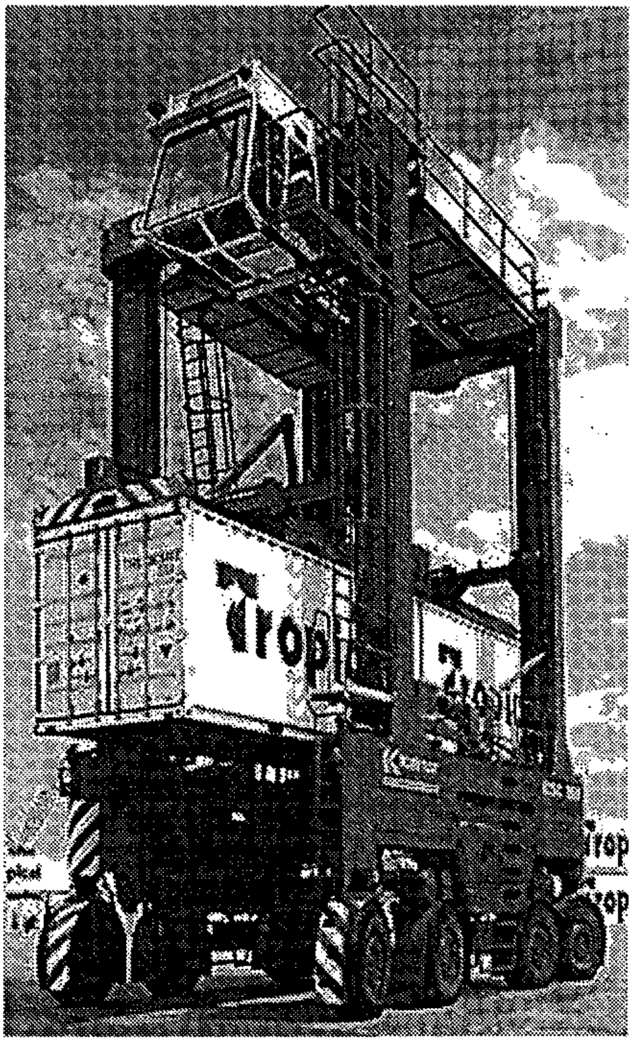

Fig. 13.15. A straddle carrier

13.7.4 Rubber-tyre gantry and/or rail-mounted gantry systems

The STS gantry crane places the containers on a terminal tractor, as shown in Fig. 13.16, or a shuttle-carrier system, as shown in Fig. 13.17 and moves the container to the stacking area where a rubber-tyre gantry (RTG) or a rail-mounted gantry (RMG) system stacks the container. The system usually stacks containers in blocks 5–9 wide and 4–6 high. The average handling capacity for one RTG crane can vary between 15–25 containers/h.

Figures 13.18 and 13.19 show details of the RTG stacking system. The system is generally economical for terminals handling more than approximately 200 000 TEUs/year. If the land for the terminal area is restricted in size or is very expensive, stacking with the RTG or the RMG system can be the only practical system for handling a large amount of containers.

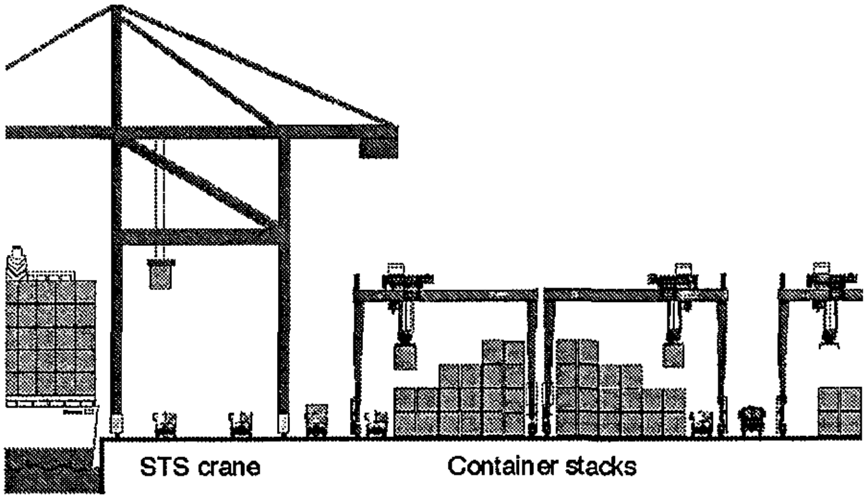

Fig. 13.16. Stacking by RTG and by using terminal tractors between the STS gantry crane and the stacking area

Generally with RTG and shuttle carriers:

(a) 2 RTG cranes and 2–3 shuttle carriers per STS crane depending on the distance between the berth and the stacking area.
(b) Good stacking density with about 800 TEUs/ha with stacking of containers 4 high.
(c) High STS crane productivity and buffer zone under the STS crane.
(d) Low labour, but high capital and medium operating costs.
(e) Low control, trucks allowed to stacking area, and efficient traffic flow difficult to arrange. If trucks are not allowed in the stacking area, one needs to increase the number of shuttle carriers by at

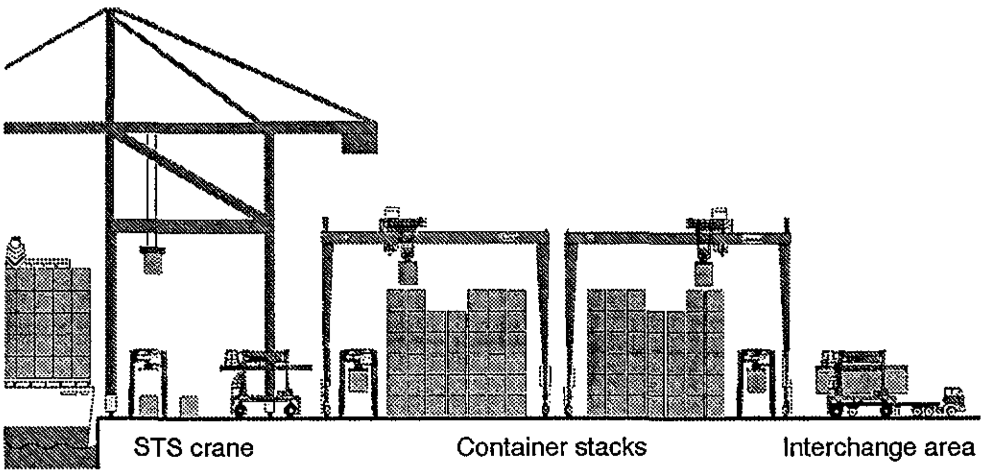

Fig. 13.17. Stacking by RTG and by using shuttle carriers between the STS gantry crane and the stacking area

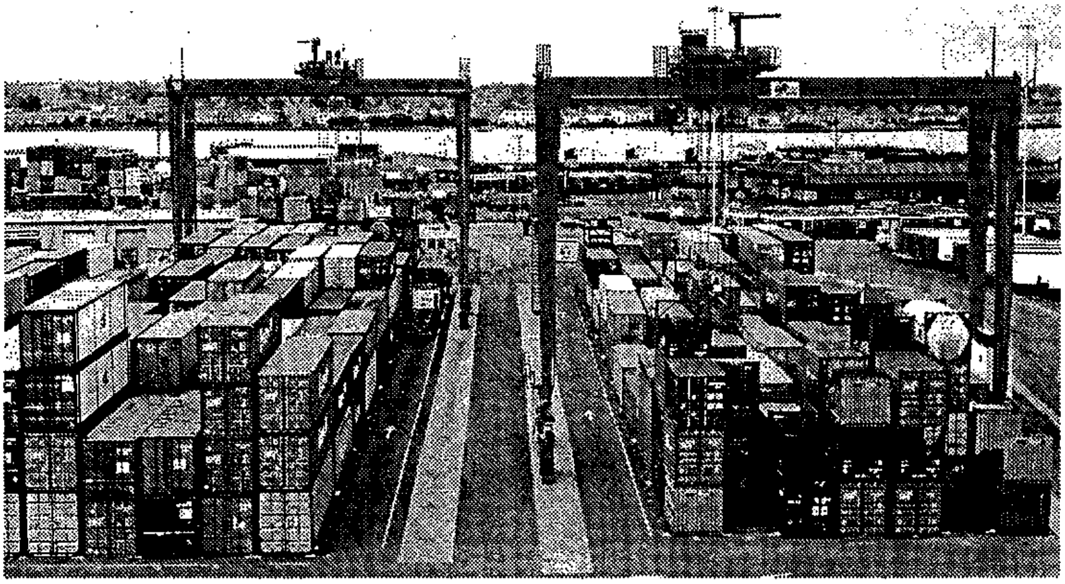

Fig. 13.18. Detail of RTG stacking 7 wide and 4 high

Fig. 13.19. Detail of RTG stacking

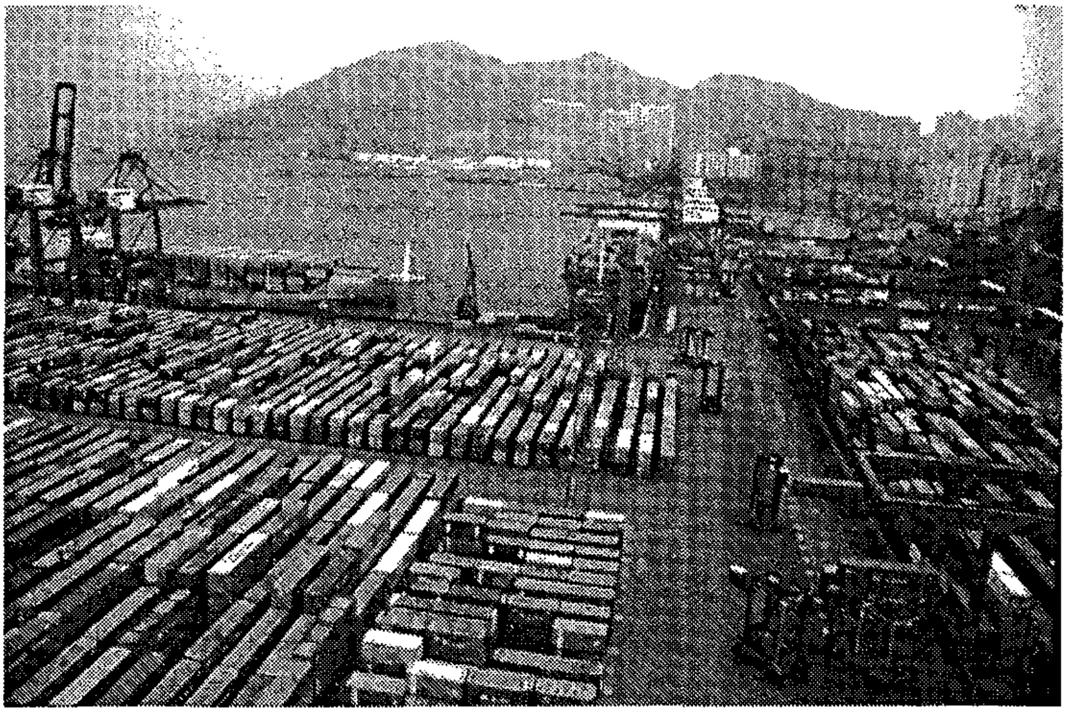

Fig. 13.20. A modern container terminal

least one. This solution gives better control over the area, but the labour and the operating costs will increase.

Generally with RTG and terminal tractors:

(a) 2 RTG cranes and 3–5 terminal tractors/STS crane depending on the distance between the berth and the stacking area.
(b) High stacking density with about 800 TEUs/ha with stacking of containers 4 high.
(c) Medium STS crane productivity and no buffer zone under the STS crane.
(d) High labour, but medium capital and operating costs.
(e) Low control, trucks allowed to stacking area.

Figure 13.20 shows a port where both the straddle-carrier and the RMG systems are used.

13.8 The terminal area requirements

The terminal area size requirement and the annual container terminal capacity will depend on, and are determined by, the choice of terminal handling equipment, operation system, available terminal area or land, and the forecast of throughput for inbound and outbound containers through the terminal. The aim for the total terminal working time should be working 24 h around the clock, 7 days a week and 365 days a year.

For pre-engineering studies the following formulas will give sufficient accuracy to determine necessary terminal areas and capacities, but for detailed design and container logistic evaluations an advanced simulation program is needed.

13.8.1 The terminal container capacity

The annual **terminal capacity** is usually expressed in terms of 20-ft container equivalent units (TEUs). The **annual container TEU movement** C_{TEU}/year:

$$C_{TEU} = \frac{A_T \times 365 \times H \times N \times L \times S}{A_{TEU} \times D \times (1 + B_f)}$$

$$= \frac{A_T \times 365 \times H}{A_{TEU} \times D \times (1 + B_f)} \times N \times L \times S$$

Or necessary **total yard area** A_T will be:

$$A_T = \frac{C_{TEU} \times D \times A_{TEU} \times (1 + B_f)}{365 \times H \times N \times L \times S}$$

$$= \frac{C_{TEU} \times D \times A_{TEU} \times (1 + B_f)}{365 \times H} \times \frac{1}{N \times L \times S}$$

$$= \frac{A_N}{N \times L \times S}$$

Where the following are the important parameters for determining the terminal capacity:

C_{TEU} = container movement/year
A_T = total yard area needed
A_N = net stacking area
H = ratio of average stacking height to maximum stacking height of the containers varying usually between 0.5–0.8. This factor will depend on the need for shifting and digging of the containers in the storage area, and the need for containers to be segregated by destination
A_{TEU} = area requirement/TEU depending on the container handling system as shown in Table 13.1
D = dwell time or average days the container stays in stacking area in transit. If no information is available, one can use

7 days for import containers and 5 days for export containers. For empty containers an average of 20-days stay in a terminal can be used

B_f = buffer storage factor in front of the storage or stacking area between 0.05 and 0.1

N = primary yard area or container stacking area compared to total yard area usually varying between 0.6–0.75 of the total yard area

L = layout factor due to shape of the terminal area varying usually between 0.7 for triangular area shape to 1.0 for rectangular area shape

S = segregation factor due to different container destinations, CMS, procedures, etc. varying usually between 0.8 to 1.0.

The area requirement A_{TEU} in m^2/TEU is dependent on the container-handling system and the stacking density, the internal layout arrangement and type of equipment used for stacking the containers, the internal-access road system, and the maximum stacking height. Recommended very approximately, the design estimates for area requirements A_{TEU}, including stacking area, internal road system, etc., are shown in Table 13.1.

Table 13.1. Area requirement/TEU

Handling equipment and method	Stacking height of container	Approximately area requirement A_{TEU} in m^2/TEU including internal roads with the following breadth or line of containers				
		1	2	5	7	9
Chassis	1	65				
FLT — front-lift truck/	1	72	72			
RS — reach stackers	2		36			
	3		24			
	4		18			
SC — straddle carriers	1 over 1	30				
	1 over 2	16				
	1 over 3	12				
RTG — rubber-tyre gantries/	1 over 2			21	18	15
RMG — rail-mounted gantries	1 over 3			14	12	10
	1 over 4			11	9	8
	1 over 5			8	7	6

The area requirement A_{TEU} in m^2 will also depend on the size of the TEU ground slot. The ground slot will usually vary between approximately 15–20 m^2 per TEU depending on the container-handling and stacking equipment.

The container stacking density is dependant on the container stacking layout (width and length), the stack height and the stack position. Therefore the arrangement of the container stacks would directly affect the accessibility and storage of the containers, and would be of central importance to the throughput and efficiency of the container terminal.

The total number of **container slots** S_L at the stacking area will be:

$$S_L = \frac{A_T \times N}{A_{TEU}}$$

Where the following are parameters for determining the total number of container slots:

S_L = total number of container slots at the stacking area
A_T = total yard area
N = primary yard area or container stacking area compare to total yard area
A_{TEU} = area requirement/TEU depending on the container-handling system.

Figure 13.21 shows the container movement/year for C_{TEU} and container storage and stacking area, A_{CP}, without the effect of N, S and L factors.

Example From Fig. 13.21 with C_{TEU} of 65 000 TEU movements/year and with an average dwell time D of 7 days in transit, the holding capacity required will be 1247 TEU. With an average area requirement A_{TEU} of 20 m^2/TEU, the net transit storage area will be 24 932 m^2. With a maximum average stacking height H of 0.8 and a later reserve area capacity R of 25 per cent, the required net stacking area A_N needing to be included for future expansion would be approximately 38 955 m^2.

If the area for container stacking compared to the total container terminal area N is, for example, 0.65, the layout factor L is 0.9, segregation factor S is 0.9 and a buffer storage factor B_f is 0.05; the total primary container yard A_T would be approximately 77 700 m^2.

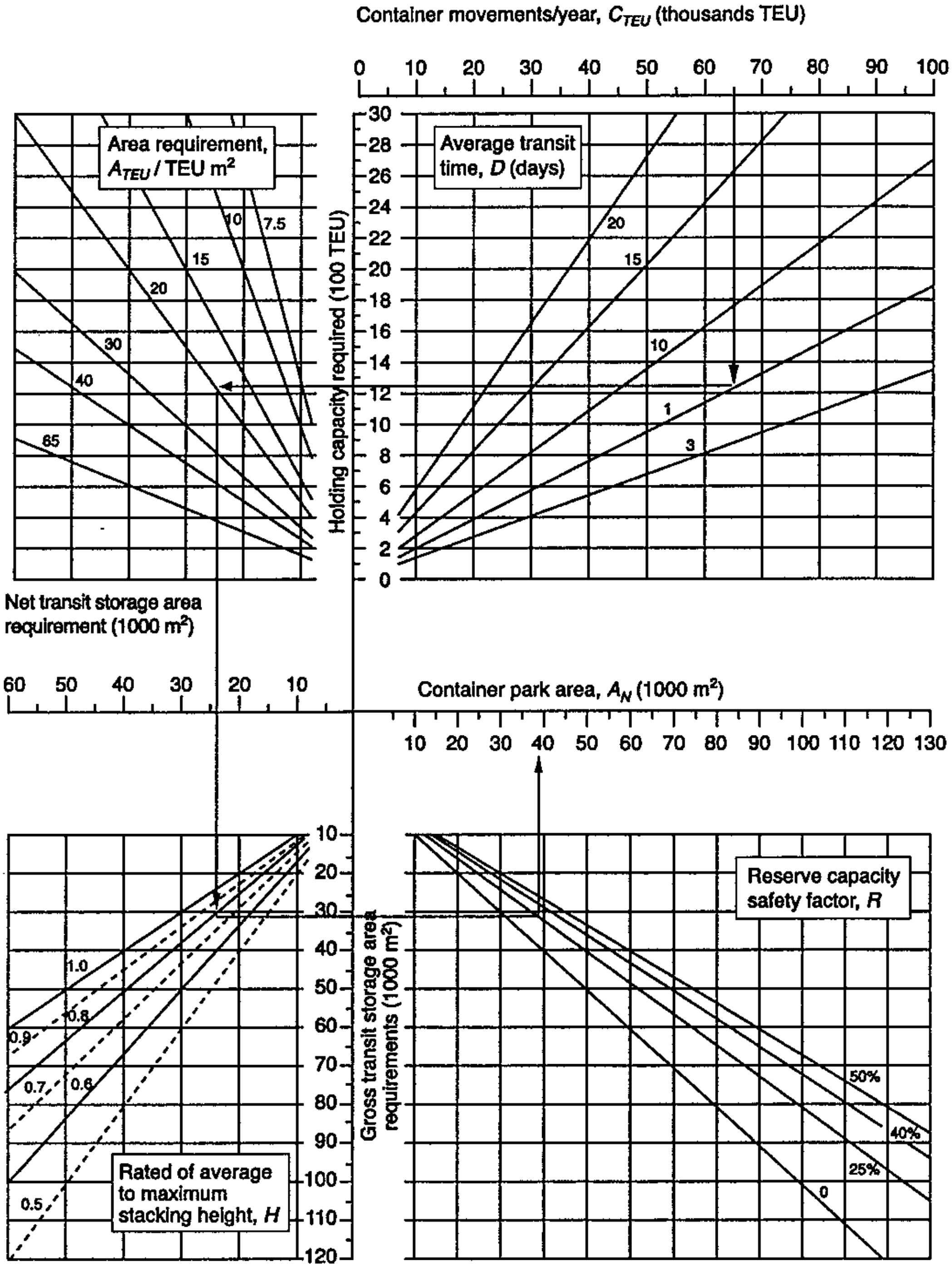

Fig. 13.21. Container storage and stacking area design diagram

13.8.2 The berth container capacity

Due to the possible stochastic arrival of the container ships/week, it is advisable to adjust the assumed container handled/week with a peak factor. The total **berth capacity** in boxes/week will be:

$$C_{BOX} = \frac{C_{TEU} \times P}{W_W \times R_{BT}}$$

Where the following are the parameters for determining the capacity/week:

C_{BOX} = container boxes handled/week
C_{TEU} = container movement/year
P = peak factor per week. Normally varying between 1.1–1.3
W_W = number of working weeks/year. Advisable to use 50 weeks/year
R_{BT} = ratio between numbers of boxes (total number of 20-ft and 40-ft containers) to number of TEU containers. Normally varying between 1.4–1.7.

The total **number of container ships** needed to berth/week including peak factor:

$$S_{CS} = \frac{C_{BOX}}{S_{BCS}}$$

Where the following are the parameters for determining the capacity/week:

S_{CS} = number of container ships berthing/week
C_{BOX} = container boxes handled/week
S_{BCS} = number of container boxes handled by one container ship.

The **working time per container ship** for loading and unloading in hours:

$$T_{WTC} = \frac{S_{BCS}}{C_N \times G_{BH} \times L_{SC} \times W_{CT}}$$

Where the following are the parameters for determining the crane working hours/week:

T_{WTC} = total working time/container ship from berthing to unberthing in hours
S_{BCS} = number of container boxes handled by one container ship
C_N = total number of STS cranes working on each container ship
G_{BH} = number of container boxes handled/container crane/h
L_{SC} = working time due to starting and closing operations due to basic output time. Normally varying between 0.8–0.95

W_{CT} = working crane time due to ship total berthing time. Normally varying between 0.7–0.9.

The total STS container **cranes working hours**/week including peak factor:

$$G_{STS} = S_{CS} \times T_{WTC}$$

Where the following are the parameters for determining the gantry crane working hours per week:

G_{STS} = total STS container cranes working hours/week including peak factor
S_{CS} = number of container ships berthing/week
T_{WTC} = total working time/container ship from berthing to unberthing in hours.

13.8.3 The berth occupancy

The arrival of shipping at port is usually a stochastic process. The number of berths required will depend on the berth occupancy. Therefore, in order to calculate the number of berths required it is essential to know if the ships arrive randomly or if there are significant peaks, such as seasonal variations, in the arrival pattern. The **berth occupancy** ratio in percentage due to working time including peak factor/week:

$$B_{OR} = \frac{T_{WTC} \times 100}{B_N \times \dfrac{W_D \times W_H}{S_{CS}}}$$

or

$$B_{OR} = \frac{G_{STS} \times 100}{B_N \times W_D \times W_H}$$

Where the following are the parameters for determining the berth occupancy ratio/week:

B_{OR} = berth occupancy ratio in percentage
T_{WTC} = total working time/container ship from berthing to unberthing in hours
B_N = number of berths
W_D = working days/week
W_H = working hours/day

Table 13.2. Berth occupancy

Number of berths	Berth occupancy factor in percentage Control of arrival of ship to berth		
	None	Average	High
1	25	35	45
2	40	45	50
3	45	50	55
4	55	60	65
5	60	65	70
6 or more	65	70	75

G_{STS} = total STS container gantry cranes working hours/week including peak factor
S_{CS} = number of container ships berthing/week.

As a rough guide, the berth occupancies for container and conventional general cargo berth operations (multi-purpose berth) should be below the figures given in Table 13.2. The figures will depend on the port administration's control of the arrival of the ship to the berth.

For oil and gas berths an occupancy factor of 60 per cent will be satisfactory, for instance for two berths.

High berth occupancy factors can seem attractive because this yields the highest berth utilization, but it is usual to assume a ratio of the average waiting time or congestion time to the average berth service time of not higher than between 5–20 per cent. The berth occupancy time will also depend on the type of berth, the type and size of the ship, transfer equipment, environmental conditions, etc.

13.8.4 Terminal capacity

As indicated below, the annual berthside crane capacity, berth productivity/m and the stacking area capacity in TEU/m^2 stacking area/year will vary considerably between the different container ports as shown:

(a) The annual berth container crane capacity varies approximately between 50 000–350 000 TEU/year, with an average of 110 000 TEU/year.
(b) The annual berth productivity/m berth front varies approximately between 500–2500 TEU/berth m per year, with an average of 1000 TEU/berth m per year.

(c) The annual container stacking area capacity in TEU/m^2 per year varies approximately between 0.5–7.0 TEU/m^2 per year, with an average of 2.0 TEU/m^2 per year.

These figures for TEU/container crane, per berth metre and per stacking area are very dangerous to use. To compare the different ports is nearly impossible to do correctly since these figures will depend on the type of container crane, the type of container stacking system used, the stacking height, the dwell time, etc. Therefore one should always, when evaluating a terminal's capacity, use the equipment, dwell time, etc. for that specific terminal and not any average figures from other terminals.

13.8.5 Hinterland

A serious restriction to the actual improvement of the STS and the terminal handling, can be the capabilities of the landside back-up system or the hinterland road and/or train system to cope with the improved efficiency of the total terminal capacity.

The number of container box passes between the terminal and the hinterland road and/or train system/working hours per day will be:

$$C_{BTH} = \frac{C_{TEU}}{R_{BT} \times W_W \times W_D \times W_H}$$

Where the following are the parameters for determining the number of boxes carried between terminal and hinterland:

C_{BTH} = number of boxes between terminal and hinterland/working hours per day
C_{TEU} = container movement/year
R_{BT} = ratio between number of boxes and number of TEU
W_W = number of working weeks/year. Advisable to use 50 weeks/year
W_D = working days/week and W_H is the working hours/day.

Whatever operating system is used, it will require close integration with the handling system between the container terminal and the associated intermodal yard and the hinterland. Improved handling at the STS interface and the terminal itself must, therefore, require the same capacity in handling between the terminal and the hinterland.

Even if the goal for all container terminals should be the possibility to work day and night, 24 h around the clock, 7 days a week and 365 days a year to reduce the pressure on, for example, the road system, experience from many large modern terminals has shown that the activity between the terminal and the hinterland road system between midnight and 4.00 a.m. is virtually more or less negligible. In practice the total traffic between the terminal and the hinterland is over approximately 10–12 h a day including a peak factor, with maximum traffic in the morning and in the afternoon. Therefore, to reduce the congestion problems the road system to the terminal would need to transfer the container distribution to other intermodal transport systems, e.g. trains.

One must, therefore, view the whole transportation system from the ship through the terminal and the gatehouses to the hinterland or *vice versa* as one operation. One should remember to provide spaces to be able to accommodate the number of lorries passing/hour through the gate, for parking the lorries in front of the gatehouses either from or out of the terminal, or from the hinterland and into the terminal.

13.9 Port security

The International Maritime Organization (IMO) will establish a new international framework of measures to enhance maritime security and through which ships and port facilities can cooperate to detect and deter acts that threaten security in the maritime transport sector. Once within the port terminal, all cargo should be capable of being identified, checked and accepted for temporary storage in a restricted area while waiting for shipment.

The port shall set the following security levels:

(a) Security level 1, normal. This is the level at which the ships and port facilities normally should operate. The security should, in principle, include fencing and guarding of the terminal, routine checking of cargo, cargo transport and storage of all cargo entering the terminal. The checking of the cargo may be accomplished by visual and physical examination, use of scanning detection equipment, mechanical devices, dogs, etc.

(b) Security level 2, heightened. This is the level applying for the period of time during which there is a heightened risk of a security incident. At this level the security is intensified to detailed checking of all cargo and operations inside the terminal.

(c) Security **level 3**, exceptional. This level can apply for the period of time when there is the probable or imminent risk of a security incident, and may include restriction or suspension of cargo movements or operation within all or part of the terminal.

All ports should have a Port Facility Security Plan (PFSP), which should indicate the operational and physical security measures the port should take to ensure that it always operates at security level 1. The security plan should indicate the additional security measures the port must take without delay to move to and operate at security level 2, and the plan should also indicate the possible preparatory actions the port should take to allow prompt response to security level 3.

13.10 The world's largest container ports

The ten largest container ports in the world in 2001 were the following:

	Port	Annual throughput in million TEU
1	Hong Kong	17.9
2	Singapore	15.5
3	Pusan	8.1
4	Kaohsiung	7.5
5	Shanghai	6.3
6	Rotterdam	5.9
7	Los Angeles	5.2
8	Shenzhen	5.0
9	Hamburg	4.7
10	Long Beach	4.4

Further reading

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14

Fenders

14.1 General

The PIANC Fender 2002 committee made the following statement: 'There is a simple reason to use fenders: it is just too expensive not to do so.'

Marine fenders provide the necessary interface between the berthing ship and the berth structure, and therefore the principal function of the fender is to transform the impact load from the berthing ship into reactions, which both the ship and berth structure can safely sustain. A properly designed fender system must therefore be able to gently stop a moving or berthing ship without damaging the ship, the berth structure or the fender. When the ship has berthed and been safely moored, the fender systems should be able and strong enough to protect the ship and the berth structure from the forces and motions caused by wind, waves, currents, tidal changes and loading or unloading of cargo. The design of fenders shall also take into account the importance of the consequences suffered by the ship and the berthing structure in the case of an eventual accident due to insufficient energy absorption fender capacity.

During the design of berth and fender constructions in the past, and even nowadays, there has been a tendency to plan and design the berth structure itself first, and only later the type of fender one hopes will satisfy the requirements as regards berth and ships. This approach to design has resulted in damages occurring quite frequently to berth and fender structures, and to a lesser degree to ships.

The correct procedure should be to plan and design the fender and berth structures jointly. The choice of fenders shall be dependent on the size of berthing ships and maximum impact energy. After having identified the fender's criteria, one can finalize the design of the berth superstructure. The following factors should therefore be considered in selecting the fender system:

(a) The fender system must have sufficient energy absorption capacity.
(b) The reaction force from the fender system does not exceed the loading capacity of the berthing system.
(c) The pressure exerted from the fender system does not exceed the ship's hull pressure capacity.
(d) The capital construction costs and maintenance costs are considered during the design of both the berth structure and fender system.

This procedure will lead to:

(a) right structural solutions
(b) lower construction costs
(c) lower annual maintenance costs.

14.2 Fender requirements

A single or easy solution to fender problems does not exist. Each type of berth structure has different demands. Factors having an impact on the choice of fender are: sizes of ships, navigation methods, location, tidal differences, water depths, etc. A ship berthing along an exposed berth structure will obviously have other demands on the fender system than if it were to berth along a sheltered berth structure.

One can talk of a berth structure's 'sensitivity' to impact from ships. Generally, a solid berth structure is more resistant to horizontal impact, whereas an open berth structure is less resistant or more sensitive. This means that a berth structure's sensitivity to berthing impact increases with its 'structural slenderness', and with increasing slenderness the fender assumes greater importance. For instance, a berth structure of concrete blocks will be less vulnerable than, for example, an open-type berth supported by piles.

When selecting a fender system, one should bear in mind the purposes of the berth structure. Structures with special functions are usually provided with fenders to accommodate certain types of ships, e.g. berths for oil tankers. But, on the other hand, if the berth should accommodate a large variation of ships sizes and types, e.g. a multi-purpose berth structure, the selection of a fender system is far more difficult and will require detailed consideration and possibly special

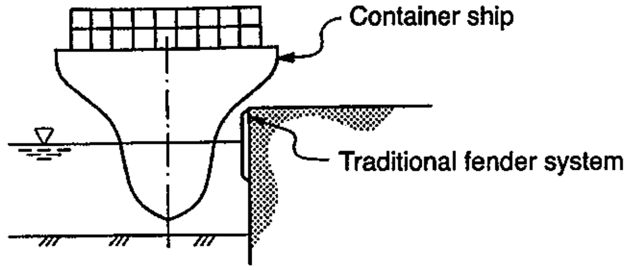


Fig. 14.1. Container ship during berthing

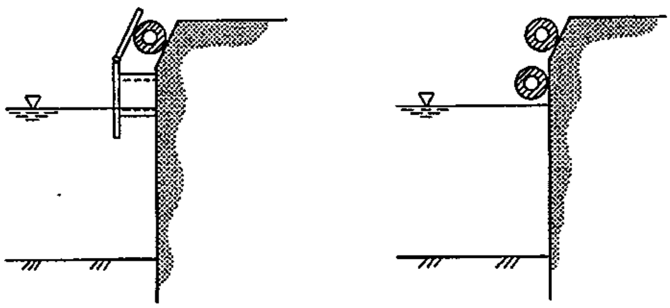


Fig. 14.2. Fenders for container ships

design treatment. The problem of selecting the right fender will be further complicated if the berth has an exposed location with difficult manoeuvring conditions and/or is subjected to extreme tidal variations.

The types of fendering provided at berth structures for general cargo ships are often unsuitable for use with specialized container ships due to their different hull and flare shapes. The large deck overhang of a container ship when berthing with an angle is illustrated in Fig. 14.1. This overhang can impose high concentrated loads on a traditional fender system because of the very small contact area between the ship hull and fender. Therefore, to solve these problems, the fender layout for a container berth should, in principle, be as illustrated in Fig. 14.2.

The horizontal distance between the berth line and the fender line should generally be kept to a practicable minimum in order to reduce, for example, the required container crane outreach as much as possible. As shown in Fig. 14.1 there must also be sufficient clearance to reduce the chance of the ship flare hitting the crane leg or the edge of the berth structure.

14.3 Surface-protecting and energy-absorbing fenders

The principal function of a fender placed between the approaching ship and the berth structure is to absorb the berthing energy or impact and transmit an acceptable load to the structure. Bearing these factors in mind, many designs of fender systems have been invented and tried out with varying degrees of success; from ordinary protecting fenders to the most sophisticated shock resistant and energy-absorbing systems.

The great differences in types of berth structures result in different requirements for the fender. Generally, a solid berth will be able to resist a high horizontal force, whereas an open-pier berth must have fenders, that absorb energy and reduce the thrust on the structure. When a ship strikes a berth structure during berthing, it has a kinetic energy that must be absorbed and which results in a horizontal force, which the berth structure must resist. In other words, when choosing a fender system, one must bear in mind the impact energy the fender must absorb, E_f, and the force P the berth structure must resist.

This means one has to choose a fender with a **fender factor**, that meets the berth requirements. The fender factor is the ratio between the force to be resisted and the energy absorption. This means that if the factor is 10 kN/kN m, a 10 kN horizontal force will be transferred to the berth structure for each kN m energy the fender absorbs. If this fender were to absorb 100 kN m energy, the resulting horizontal force to be resisted by the berth will be 1000 kN. The ideal fender is one that will absorb large amounts of kinetic energy and will transmit low reactive loads into the berth structure and the ship hull.

Fenders can generally be divided into two groups:

(a) **Surface-protecting fenders** that transmit a high impact or reaction force to the berth structure for each kN m energy absorbed, i.e. the fender factor P/E_f is high.
(b) **Energy-absorbing fenders** that transmit a low impact or reactor force to the berth structure for each kN m energy absorbed, i.e. the fender factor P/E_f is low. From the performance point of view, the low reaction and the high energy absorption with constant reaction over part of the deflection range are a distinct advantage, but for a large range of ships the fenders can be too hard for the smaller ship using the berth.

When checking manufacturers' catalogues to see if, for example, a cylindrical fender is of the type comprising surface-protecting or energy-absorbing fenders, one should bear in mind that the capacity of such a fender is given when about 50 per cent compressed.

As regards the force P, there are two factors, which decide its magnitude and, consequently, the type of fender to be chosen:

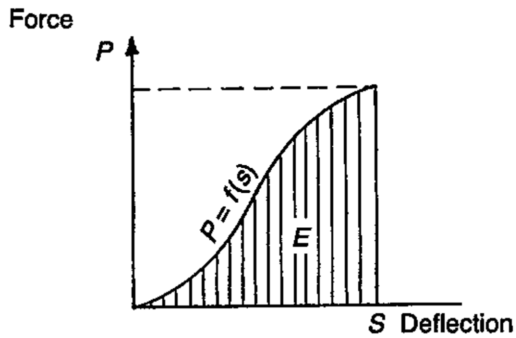


Fig. 14.3. General working diagram for fenders

(a) the horizontal force the berth structure can resist
(b) the maximum pressure the ship's side can withstand.

On the whole, the horizontal force on the berth structure is the decisive factor where smaller ships are concerned and for larger ships it is the pressure on the ship's side. The latter depends, of course, on the contact area available for the pressure distribution between berth and ship's side during berthing.

From a berth designer's point of view the purpose of a fender system is to reduce the reaction force and transmit a designed thrust to the berth structure, that it can bear without difficulty. On the other hand, the impact energy increases with the ship displacement to a greater extent than the strength of the hull does. Therefore, the ship's hull also needs energy-absorbing fenders. Similar requirements for impact-reducing fender systems also arise where the berth structure is exposed to difficult berthing and weather conditions. In other words, the ideal aim for a fender system is to be able to absorb high-impact energy and transmit a low reaction force to the ship's hull and the berth structure.

Each type of fender system has its own characteristic force/deflection curve, which is shown in principle in Fig. 14.3. The area under the curve represents the total energy absorbed in deflecting the fender. The shape of the curve gives an idea of the fender's energy efficiency and the impact intensity. Generally, due to fender characteristics, one can distinguish three different curve patterns for the main types of fender on the market nowadays, as shown in Fig. 14.4. Fender 1 is characterized as a hard fender, fender 2 as a medium fender and fender 3 as a soft fender. This is illustrated in Fig. 14.5, where the areas under each of the curves are equal (Area A = Area B = Area C). The different fenders (fenders 1, 2 and 3) have the same design reaction force and the same energy-absorption capacity, but different deflection.

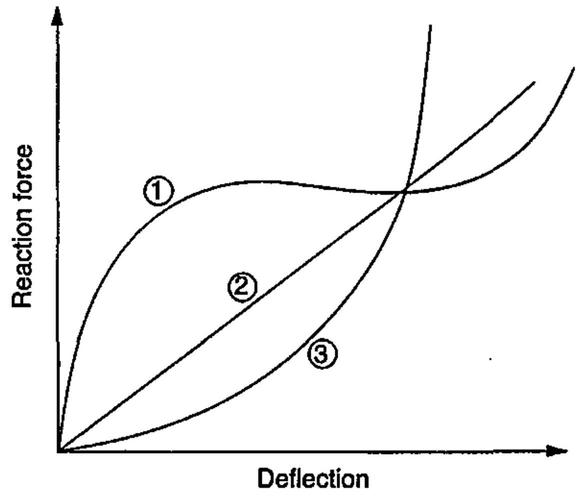


Fig. 14.4. Working diagram for three different types of fenders

It is evident from Fig. 14.5 that fender type 1, or the buckling type fenders, e.g. cell fenders, requires considerably less deflection to absorb the design energy than a side-loaded cylindrical rubber fender. The characteristics of fender type 1 causes the maximum reaction force to occur during almost every berthing, even with ships smaller than the maximum design ship. Therefore, due to rather high contact pressures against the ship's hull, a panel or fender wall between the ship hull and the fender itself is often needed to reduce this contact pressure. The fender type 1 has, as illustrated in Fig. 14.5, a higher performance from the energy point of view but may not be recommended when the tonnage range of the ships likely to berth entails a very wide range of energies to be absorbed. The buckling of fender type 1 is also susceptible to significant reduction in energy-absorption capacity when subjected to impacts not perpendicular to the fender face.

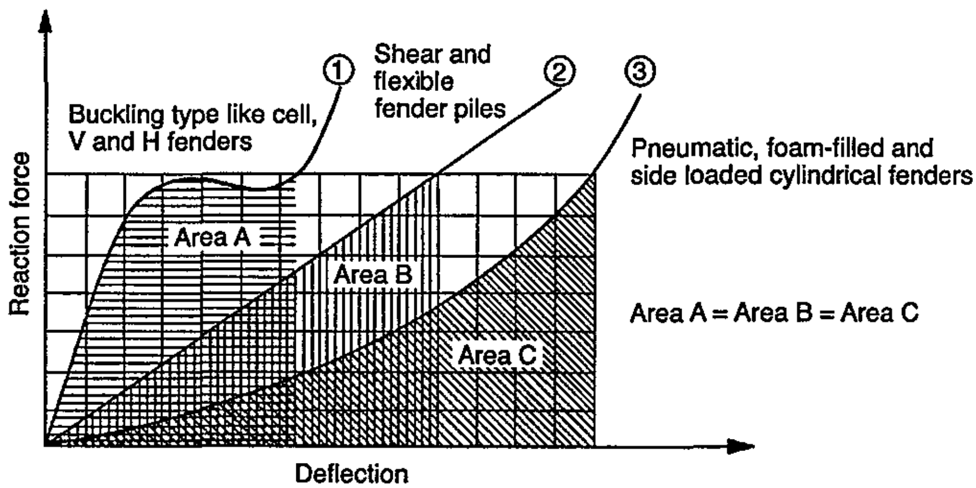


Fig. 14.5. Reaction/deflection characteristics of various fender types

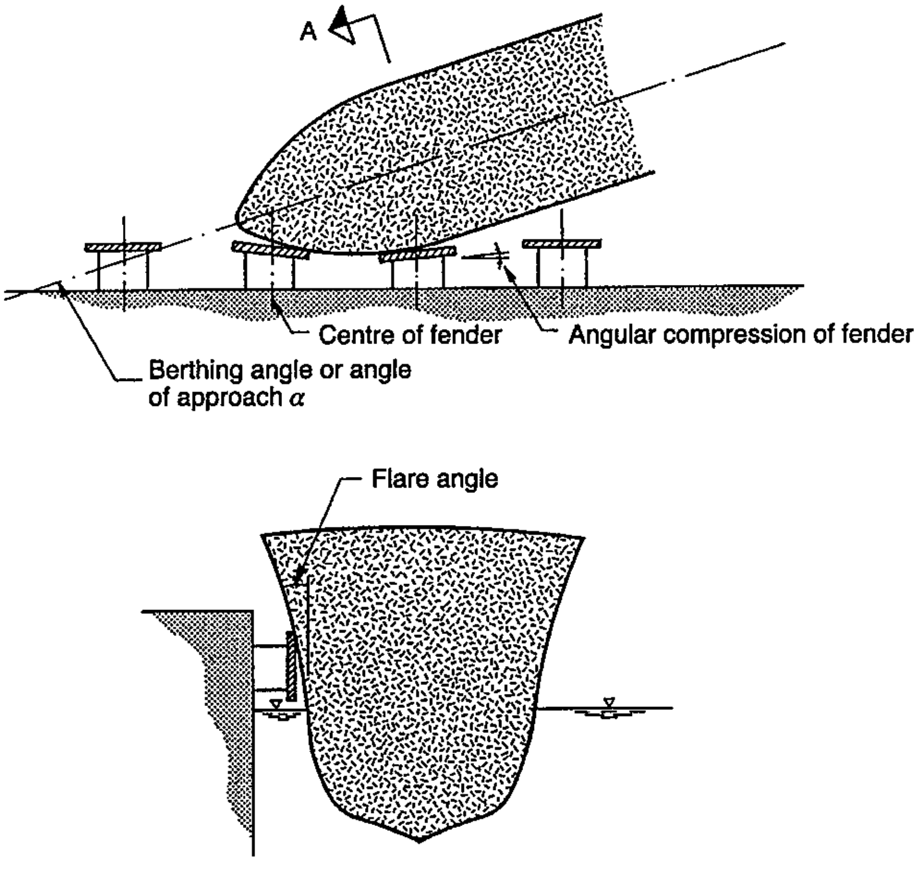


Fig. 14.6. Angular compression of fender

The flexible fender piles, or fenders type 2, are often an alternative where the soil conditions are suitable because they can combine the functions of a fender and breasting structure. The fenders type 3 or the soft fenders are very popular where energy absorption requirements are not too high, but as can be seen from Fig. 14.5, they must be larger than the corresponding fender type 1 and thus require greater reach of the cargo-handling equipment.

After having estimated the energy to be absorbed by the fenders, the reaction forces against the berth structure can be read from the fender manufacturer's curves. If the fender manufacturer has not stated a tolerance on the figures quoted for reactions and energies, a tolerance of +10 per cent should be taken into account in the design of the fender system. These curves are usually based on uniform deflection of the fenders. As shown in Fig. 14.6, a non-uniform deflection of the fender system can occur due to the following:

(a) the angle of approach between the ship and the fender line

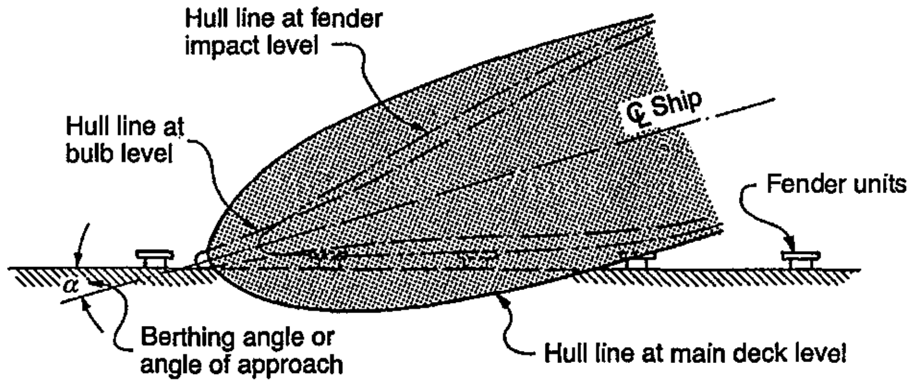


Fig. 14.7. Ship hull curves

(b) the curve of the ship hull in plan where the ship makes contact with the fender .
(c) the flare angle of the hull in section.

A study of a ship's hull curves will, as illustrated in Fig. 14.7, show that the contact angles and the contact point will vary with the angle of approach and the height of the fender relative to the ship, as also shown in Fig. 14.8. The designer should, therefore, establish a maximum safe value of the berthing angle or angle of approach that can be economically achieved due to the ship hull, flare angle and bulbous bow having regarded both the fender and the berth structure layout.

Previously the flare angle of the shipside was not considered since most of the ships had nearly vertical shipside at the contact point between the ship and the fender. But for modern ships, like the third generation or larger container ship, one has to consider the flare angle under the selection of a fender system. The little available data of the flare angle at the point of contact with the fenders show that there is a wide range of flare angles for a given approach angle, both within a particular ship category and especially between different types of ships.

For general cargo ships the flare angle at an approach angle of 5° is about 8–15° and for an approach angle of 10° is about 15–25°. For container ships the flare angle at an approach angle of 5° is about 10–16°, and for an approach angle of 10° it is about 20° up to 40°. It appears that general cargo ships, bulk carriers and tankers have less variation in the flare angle than container ships. The ships that have less flare angle seem to have the largest block coefficients.

From these flare angle figures it is shown that container ships exhibit greater flare angle than general cargo ships. In the future this will

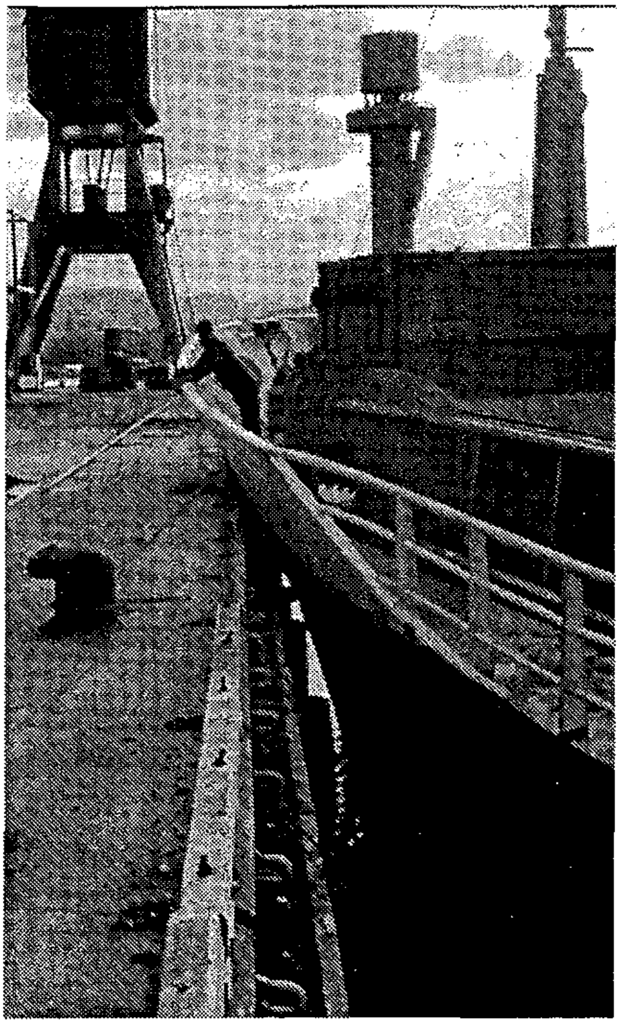


Photo 14.8. Contact between ship and berth

probably govern the design and layout of the fenders in commercial ports. Consideration should also be given to ships with a high block coefficient like bulk carriers which, because of their higher block coefficient, will have a hull with a small radius in plan at the point of contact between the ship and the fender, and will therefore require a closer spacing between the fenders.

The choice of fender type will, in many cases, determine the design of the berth superstructure. Generally the protective fender demands a solid berth structure because of the large force it exerts against it. In other words, a cheap fender system and an expensive berth construction. On the other hand, an energy-absorbing fender imparts a lesser force to the structure, thus demanding a less solid berth construction, which generally means an expensive fender and a cheaper berth construction.

These parameters will not only apply to new berth constructions, but very much so to old berth structures being upgraded. Nowadays there are many old berth structures still in use with sufficient water depth in front of the structure, but because of their structural design and their protective fenders of, for example, wood materials, they will not be able to accommodate modern ships of increasing dimensions which exert greater horizontal loads than the structures were designed for. However, if energy-absorbing fenders replace the protective fenders, the structures will, in most cases, be able to also accommodate these larger ships.

14.4 Different types of fenders

In the following, the various types of rubber fenders will be discussed, of which nowadays the most used are prefabricated fenders. Since the first rubber fenders were made, in the 1930s, they have proved resistant to aggressive and polluted water as well as wear and tear from ships, as long as they have been correctly installed. Their purchase price and maintenance costs are also below those of most other types of fenders. Rubber fenders are produced in many sizes and shapes, depending on their function. One should be aware of the fact that for different manufacturers producing apparently identical fenders, their fender factors may differ entirely.

Basically there are two types of fender:

(a) Fenders which in principle are fixed or mounted to the berth structure. The fixed fenders are again subdivided into the buckling fenders (cell fenders, V-type fenders, etc.) and the non-buckling fenders (cylindrical fenders).
(b) Floating fenders between the ship and the berth structure. The floating fenders are again sub-divided into the pneumatic fenders and the foam-filled fenders.

Figure 14.9 lists different types of rubber fenders and Fig. 14.10 indicates whether they are mainly surface protecting or mainly energy absorbing. As can be seen, for example, the different sizes of cylindrical fenders have under radial loading fender factors P/E_f varying from about 25 kN/kN m to about 1.3 kN/kN m.

There is not necessarily any connection between the fender factor P/E_f and the flexibility or the rigidity of a fender. There are fenders with low fender factors (energy-absorbing fenders), which are very rigid, and fenders with high fender factors (surface-protecting

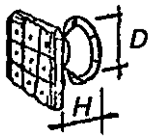
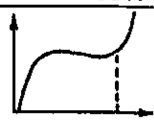
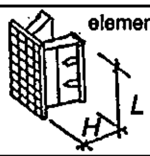
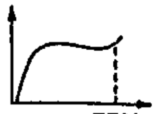
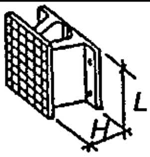
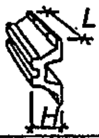
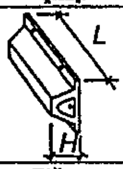
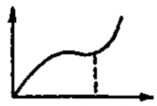
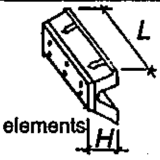
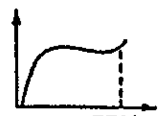
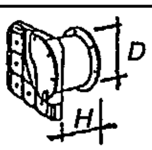
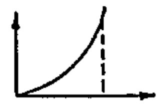
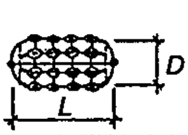
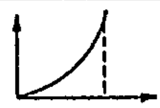
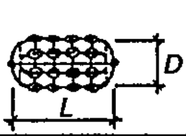
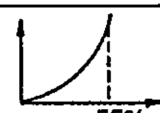
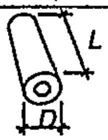
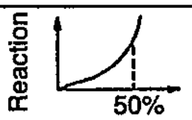
Type	Fender shape	Sizes in mm	Reaction kN	Energy kN m	Performance curve
Circular shape of the buckling fender with panel contact		d/D/L 295/500/300 ↓ 1765/ 2880/1800	60 ↓ 3775	9 ↓ 3530	 65%
		D/H 400/550 ↓ 3000/3250	52 ↓ 5800	8 ↓ 6700	 47·5 and 52·5%
Longitudinal shape of the buckling fender with panel contact		H/L 300/600 ↓ 1800/2000	66 ↓ 1708	9 ↓ 1260	 55%
		H/L 400/500 ↓ 2500/4000	140 ↓ 6900	22 ↓ 7000	 50, 52·5 and 60%
V-type		H/L 250/1000 ↓ 1000/2000	150 ↓ 2290	15 ↓ 940	 50%
		H/L 200/1000 ↓ 1300/3500	150 ↓ 3400	10 ↓ 1500	 45%
		H/L 300/600 ↓ 1800/2000	66 ↓ 1708	9 ↓ 1260	 55%
Airblock		D/H 600/450 ↓ 3200/3200	138 ↓ 6210	15 ↓ 4990	 60 and 65%
Pneumatic		D/L 500/1000 ↓ 4500/12 000	50 ↓ 10 570	4 ↓ 9080	 60%
Foam filled		D/L 1000/1500 ↓ 3500/8000	200 ↓ 4050	41 ↓ 3000	 55%
Cylindrical		D/L 150/1000 ↓ 2800/5800	80 ↓ 6600	3 ↓ 5000	 50% Rated compression

Fig. 14.9. Different types of rubber fenders

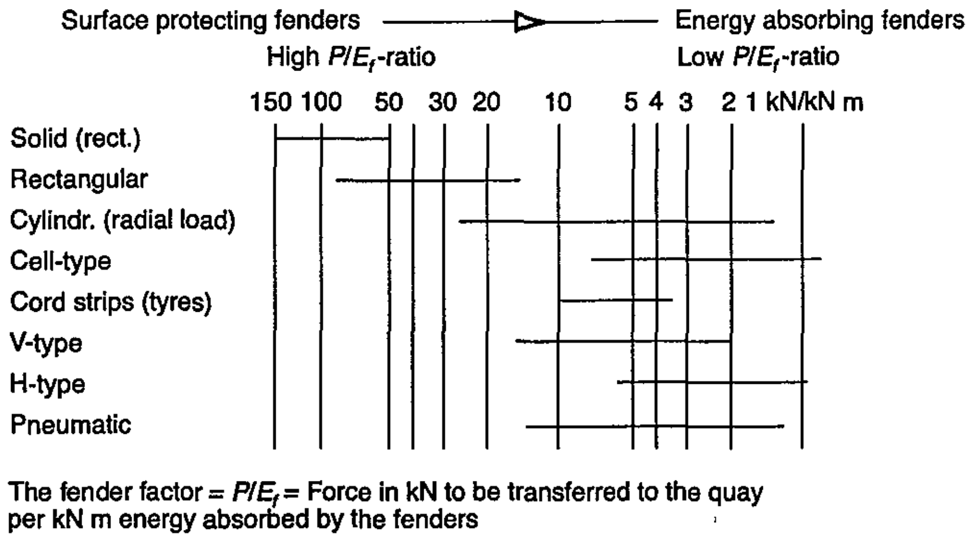


Fig. 14.10. Fender factor for different types of rubber fender

fenders), which are flexible. For instance, old car tyres used as fenders are very flexible but act as surface-protecting fenders. Even under small loads they are pressed flat and function only as solid fenders.

Most of the characteristics for the different fender types shown in Fig. 14.9 are based on data published by the different fender manufacturers. The actual fender performance may vary by as much as ±10 per cent and the characteristics are based on normal or perpendicular impacts against the fender. The fender performance may vary considerably when subjected to angular impacts, which is the most common case.

Therefore, based on the manufacturers' performance curves, due to the manufacturing tolerance, it is usual to recommend a tolerance of −10 per cent for the energy absorption and +10 per cent for the maximum reaction forces on the catalogue performance figures. This reaction force is a characteristic load, which should be used for the design of the berth structure.

14.5 Installation

The installation and mounting of the fender systems should be of robust and simple design, and only one type of metal should be used to avoid electrochemical corrosion, and no mountings should be allowed to touch the steel reinforcement in the concrete. This is especially important if the fenders are mounted on structures with cathodic protection against corrosion.

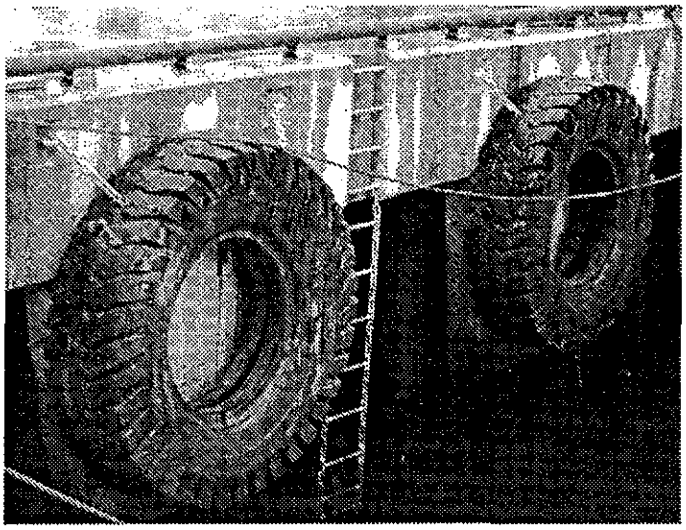


Fig. 14.11. Fenders of old rubber tyre

The most used fender system around the world is the installation of old used rubber tyres in front of the berth structures, as shown in Figs 14.11 and 14.12.

Cylindrical rubber fenders are the second most used fender system around the world. They are manufactured with outside diameters ranging from about 15 cm to approximately 2.6 m. The fender factor for a 40 cm diameter radial load will be 20 kN/kN m and for a 1 m diameter radial load 5 kN/kN m. Figure 14.13 shows different ways of installing cylindrical fenders, which are the most common type of

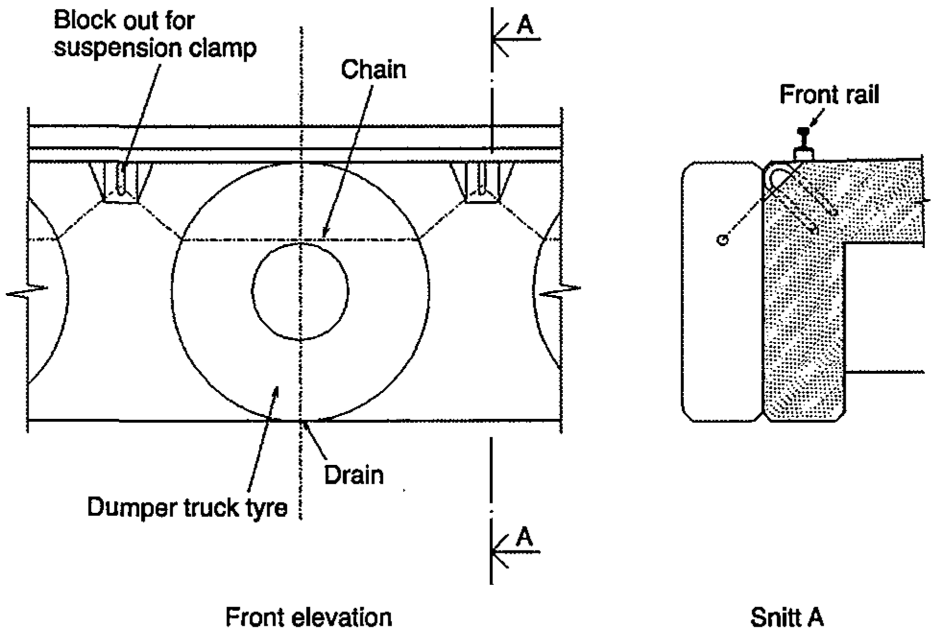


Fig. 14.12. Installation of old rubber-tyre fender

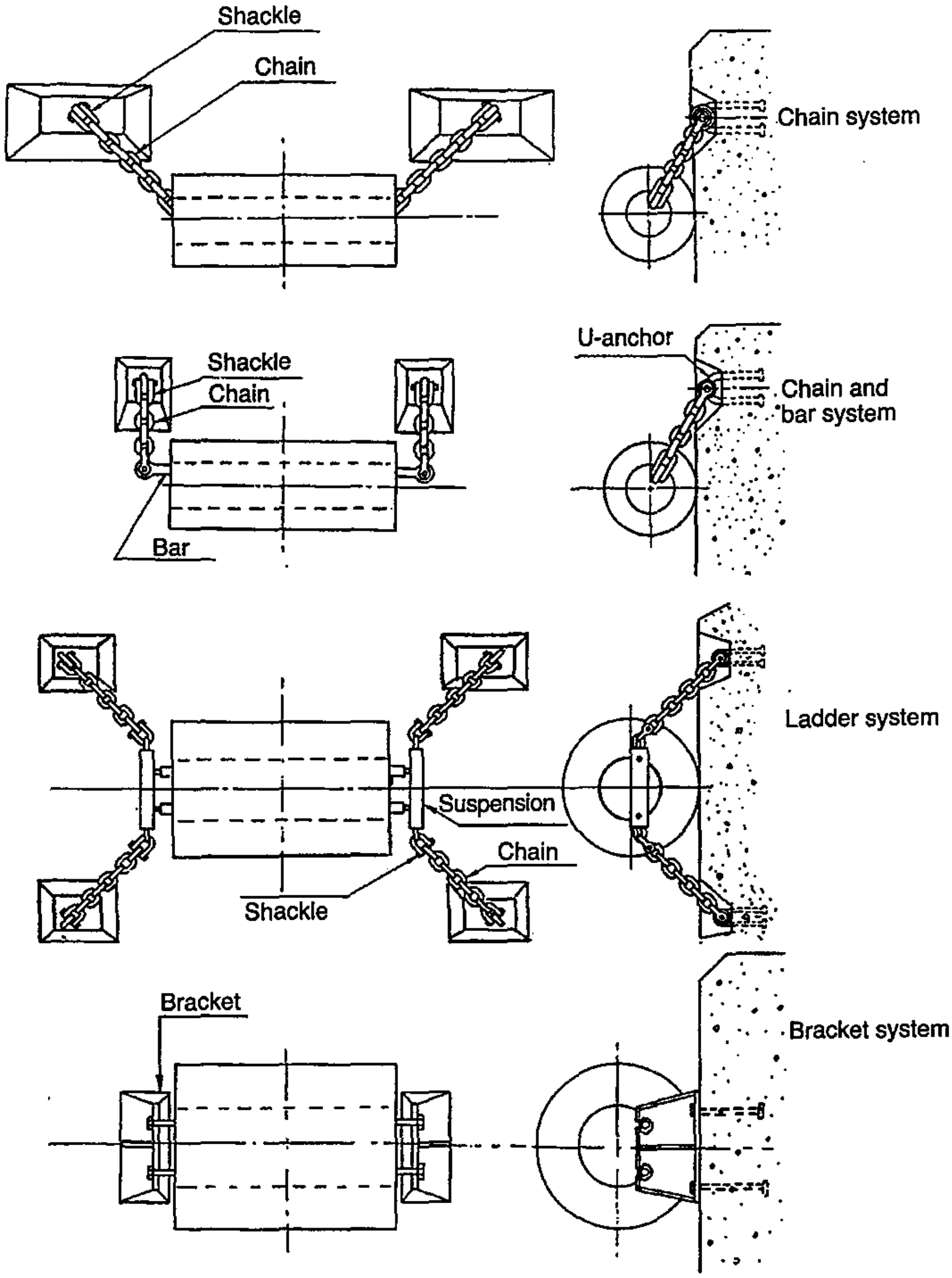


Fig. 14.13. Different ways of installing cylindrical fenders

fender. The ladder and bracket installation systems are used for larger ships. Figure 14.14 shows a large cylindrical fender on a breasting dolphin.

Pneumatic floating fenders are available in sizes ranging from 50 cm outside diameter (OD) and 1.0 m length, to 4.5 m OD and 12 m length. They are well suited as buffers between two tankers or between a tanker and a berth structure.

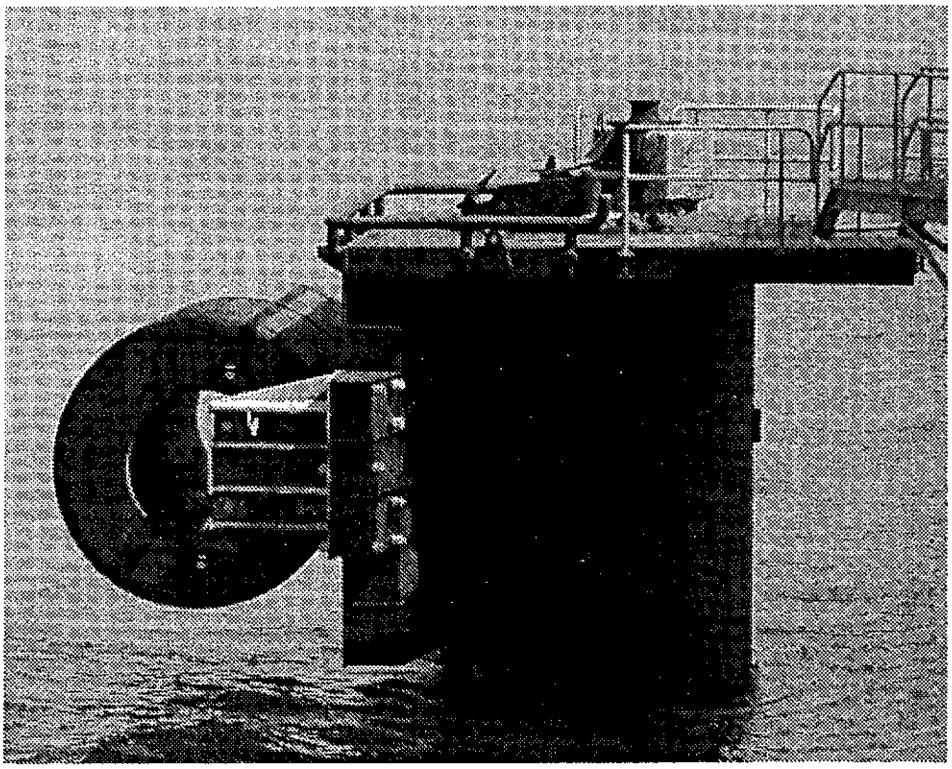


Fig. 14.14. Large cylindrical fender on a breasting dolphin

14.6 Effects of fender compression

After having calculated the probable impact energy a ship will have when berthing, one can deduce from the manufacturers' catalogues the compression of the various fenders and the thrust the latter will transmit to the structure. Manufacturers always provide two diagrams for fenders, one showing the relationship between energy and compression and the other the impact force and compression relationship.

In Fig. 14.15 two such diagrams, both for a buckling fender and a side-loaded cylindrical fender, have been combined to illustrate what happens to the two different fender types when a ship is berthing. As a more detailed illustration, the cylindrical rubber fender with 1500 mm OD and 800 mm inside diameter (ID) and a 1500 mm length will, with 50 per cent compression, absorb impact energy of 330 kN m. The resulting force to be resisted by the berth structure will be 900 kN with a fender factor P/E_f 900/330 = 2.7 kN/kN m. What is interesting about the large fenders which are designed for bigger ships, is that they have a high fender factor with low compression, at 10 per cent, the $P/E_f = 14.0$ kN/kN m. Where smaller ships are concerned, they will have little energy-absorbing effect but function more as surface-protecting fenders. The curve shows that the fender factor decreases with increasing

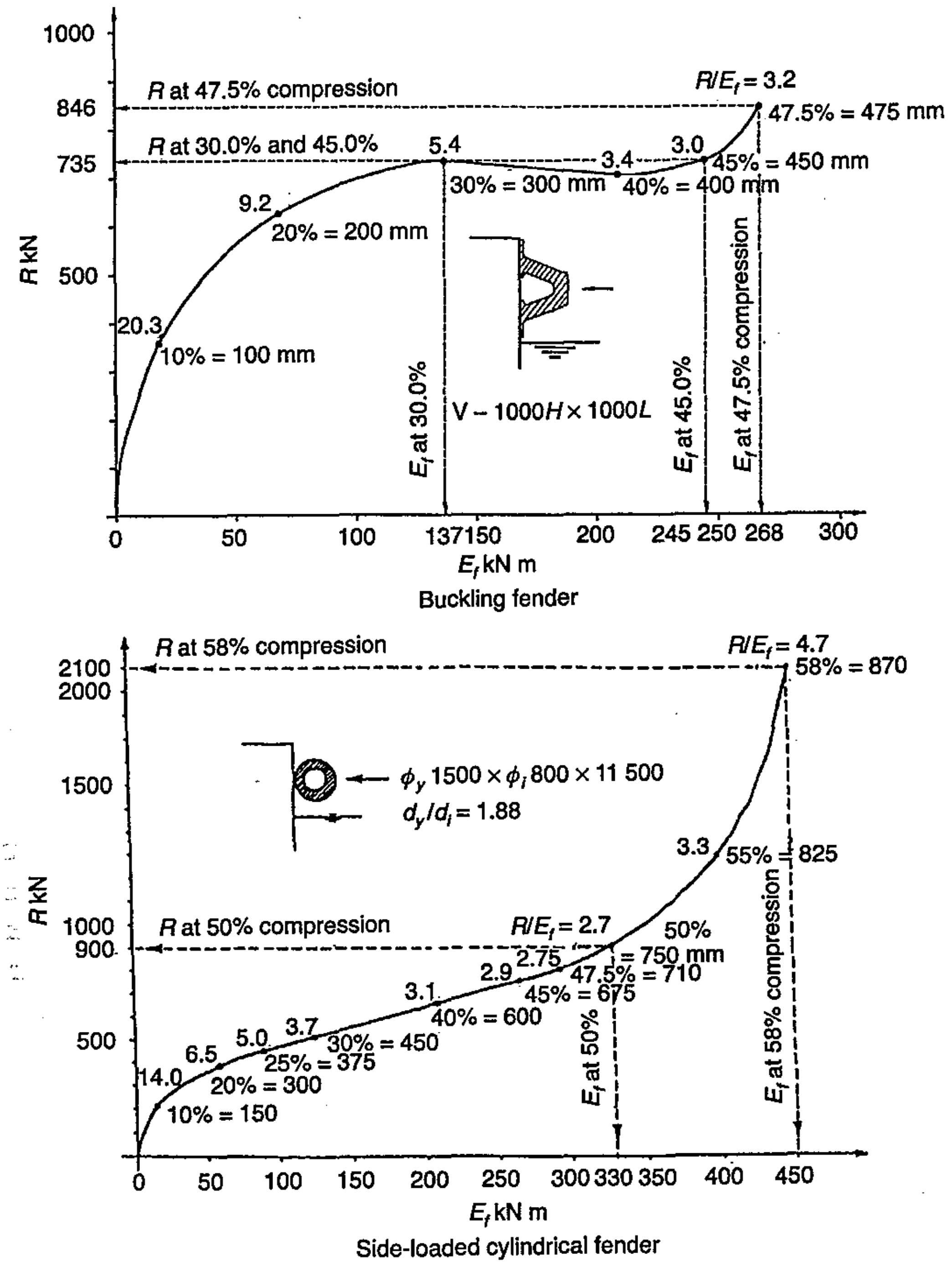


Fig. 14.15. The effects achieved at various degrees of compression of a buckling fender and a side-loaded cylindrical fender

compression, to as much as 50 per cent when it is 2.7 kN/kN m. Beyond this the factor increases with increasing compression.

It must be realized that for both fender types in Fig. 14.15, the fenders can absorb more energy even beyond approximately 50 per cent compression, but the force to be resisted by the berth structure will then increase excessively. This is due to the fact that the fenders

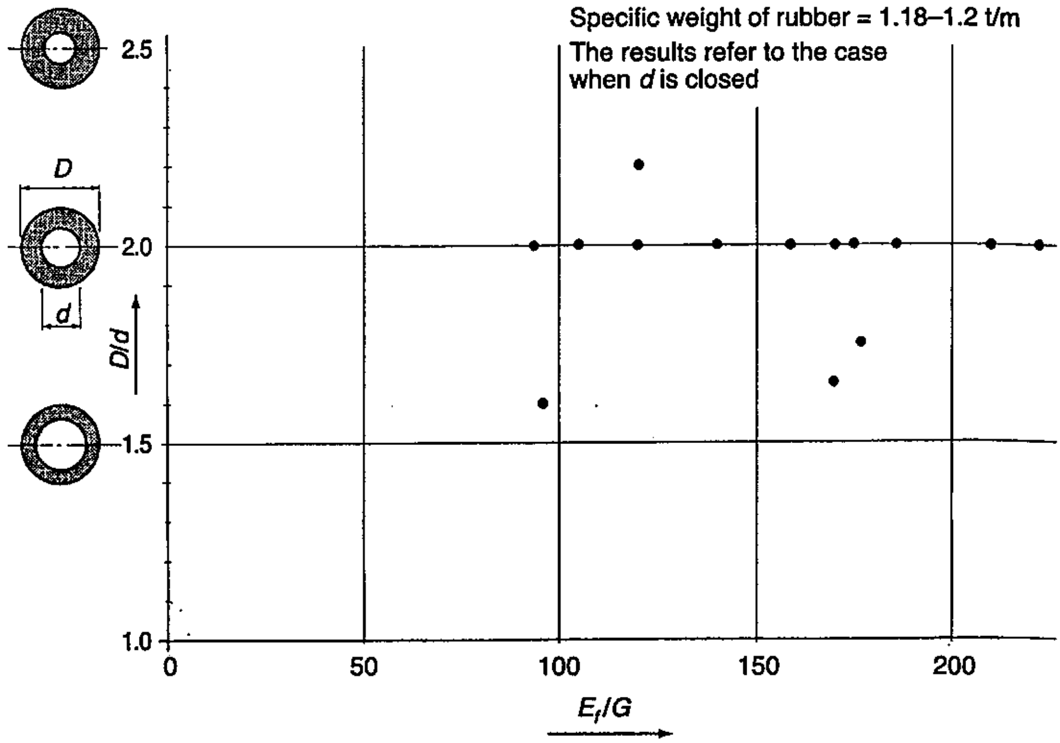


Fig. 14.16. Energy absorption related to diameter ratio to ton weight of fender

have been compressed to such an extent that they now function more like a surface-protecting fender. As a fender unit can only absorb a fixed amount of energy before failure, the fender structure can be provided with a device or an overload collapsible unit to prevent overload of the fender. The collapsible unit can be constructed in either concrete or steel, and installed between the fender and the berth structure. To prevent failure or damage to the fender, the collapsible unit can be designed to collapse for a reaction force equal to the fender reaction at about 55–60 per cent compression of the fender.

The relationship between OD and ID for a cylindrical rubber fender under radial load has a great influence on the fender factor. The usual ratio OD/ID is 2, but some manufacturers can produce fenders down to a ratio of about 1.75. If one can choose between several fenders with the same OD, the fender with the smallest diameter ratio will usually have the lowest fender factor and also be more economical. Figure 14.16 shows, for different cylindrical fenders, some results from different manufacturers showing the energy absorption of the fender related to the diameter ratio to ton weight of the fender. As shown with a diameter ratio of 2.0, the energy absorption to ton weight of fender will lie between about 90–210 kN m/ton of fender. As manufactured cylindrical rubber fenders are usually paid for by weight of rubber, one should look for a fender

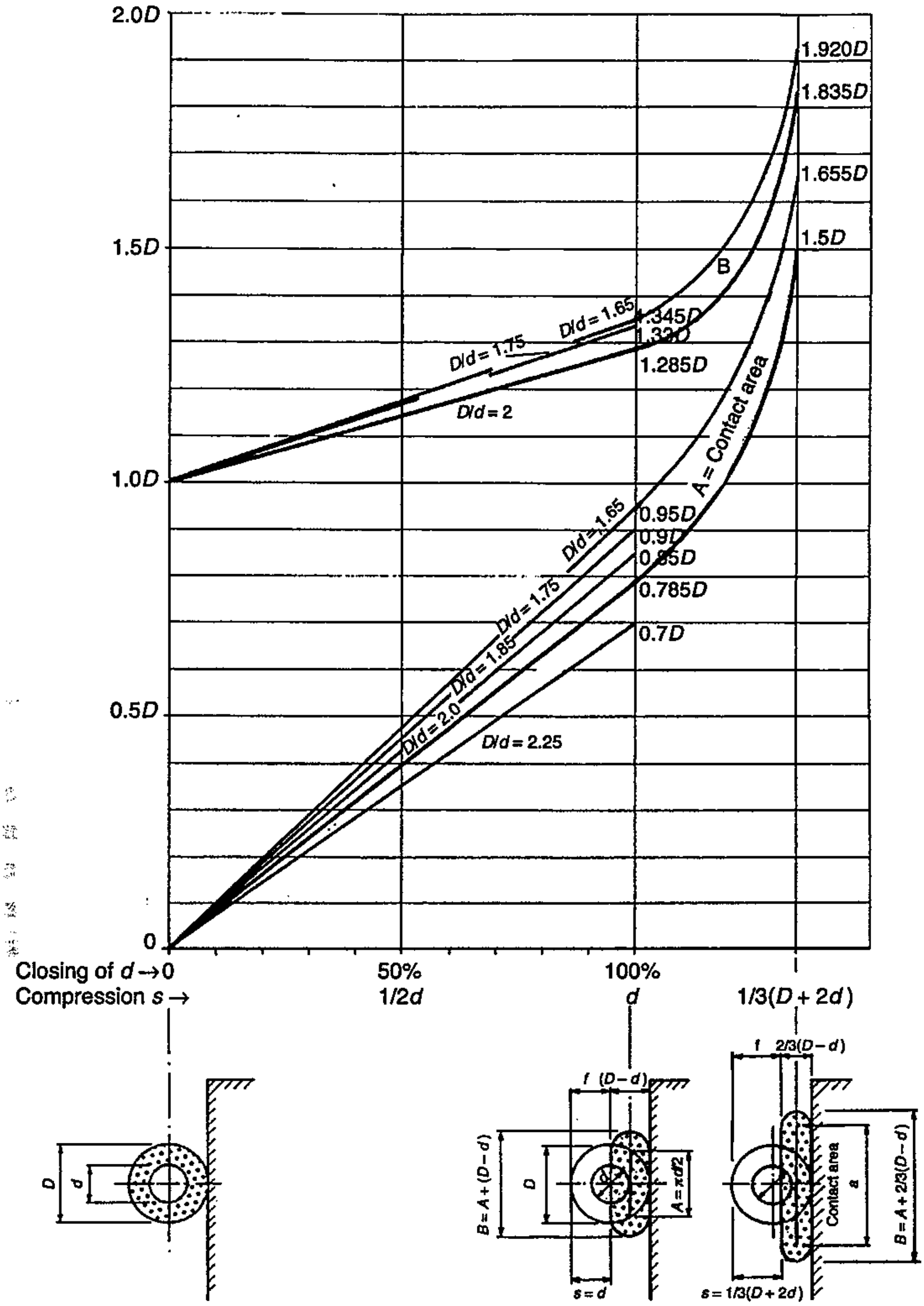


Fig. 14.17. Fender contact area

with an energy absorbed/weight ratio of at least about 160 kN m/t for cylindrical fenders. Furthermore, the fender with the lowest diameter ratio will have the lowest surface pressure, in kN/m^2, between fender and ship's hull, as illustrated in Fig. 14.17.

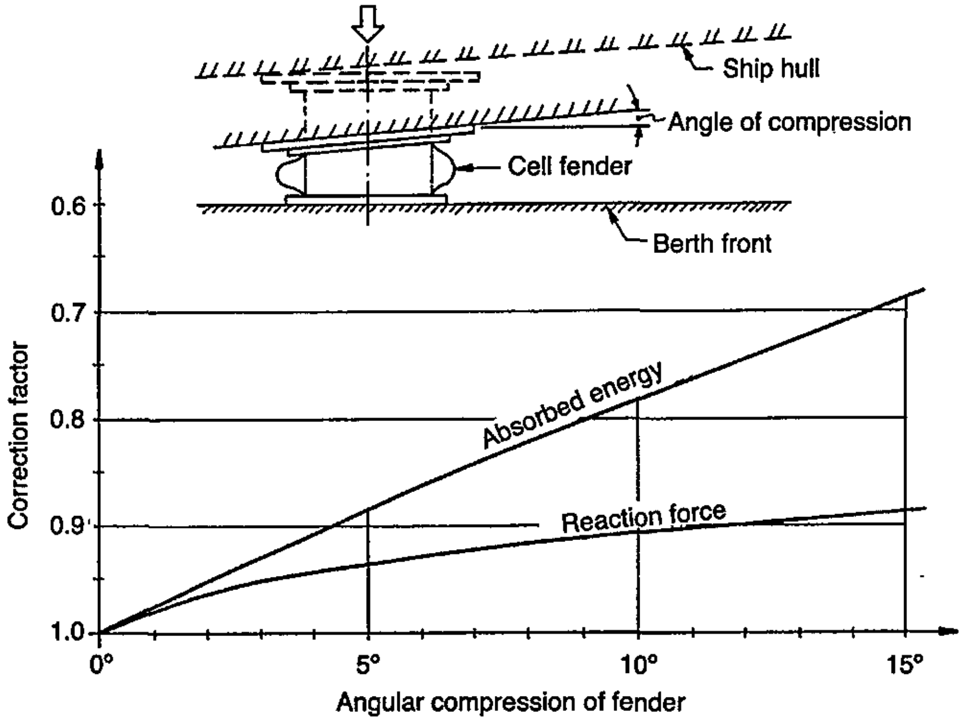


Fig. 14.18. Correction factor under angular compression

The performance of a fender during angular loading is illustrated in Fig. 14.18. During the actual berthing conditions a fender will generally be loaded or compressed at an angle more or less equal to the approach angle of the berthing ship. This angular compression of the fender will change the characteristics of the fender reaction force and absorbed energy, compared to a fender compressed normal to the berth front. The correction factor for a single-cell fender unit compressed under different angles is shown in Fig. 14.18. Therefore, the choice of fender will depend on the angular compression of the fender due to the curve of the ship's hull in plan and the flare angle.

14.7 Properties of a fender

In order to select the most proper and suitable fender for a particular berth, it is important to know the performance characteristics and the properties of each rubber fender type. Below, some of the more important factors will be discussed.

14.7.1 Design life

Although most rubber fender manufacturers produce fenders with a design life of about 20–30 years, the actual life of the fender will depend on the type of ships, the frequency of berthing and the influence of the natural environment, such as temperature, ozone density, sunlight hours and intensity, pollution and salt water as well as oils and fats. But according to the manufacturers, most damage to fenders is the result of the fender being either under-dimensioned, bad manoeuvring or too high berthing velocities. For this reason a higher safety factor should be used in designing the fender system. The different rubber manufacturers recommend a design life of about 5–15 years for fender systems installed on general cargo berths with a large range of different types of ships, and a design life of about 10–20 years for more particular berths, such as an oil berth.

14.7.2 Fender testing

The load deflection and energy deflection characteristics of a fender are only valid if the fender has been preconditioned by compression to the rated values at least three times before use. If not, the first maximum compression produced by the ship may well give higher than expected reactions. If the fenders are made of laminated rubber the strength of the lamination shall be equal to that of the material itself.

After PIANC Fender 2002, the break-in deflection of the actual fender element should be at least the manufacturer-rated deflection, and at least one cycle should be performed. The break-in deflection should be mandatory for all fender types with a catalogue reaction rating of 100 tonnes or more, and they should be installed on a pile supported berth structure. For other fenders installations the customer should stipulate the break-in deflection.

The manufacturers' published performance curves and/or tables of the Rated Performance Data (RPD) should be based on one of the following PIANC testing requirements:

(a) The traditional and widely used constant velocity (CV) method with constant slow velocity deflection. This method is the preferred method of a majority of manufacturers.
(b) The decreasing velocity (DV) method. Initial berthing deflection velocity of 0.15 m/s decreasing to no more than 0.005 m/s at the end of the test.

The RPD should also be based on:

(a) testing of fully broken-in fenders
(b) testing of fenders stabilized at 23 °C ± 5 °C

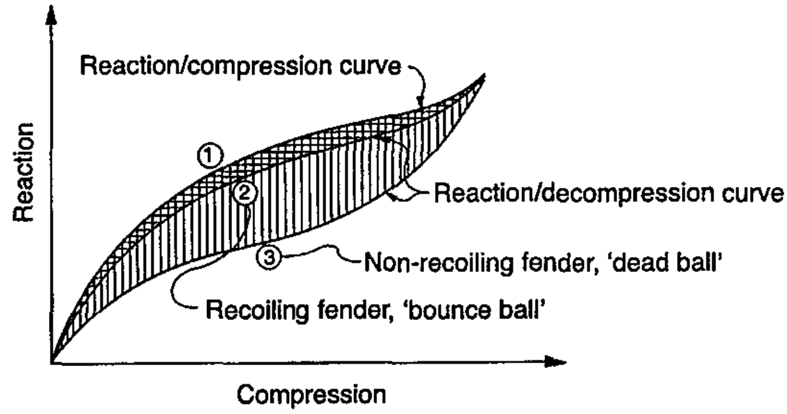


Fig. 14.19. Hysteresis effect

(c) testing of fenders at zero degrees angle of approach
(d) deflection (berthing) frequency of not less than 1 h.

14.7.3 Hysteresis

The berthing energy or receivable energy which must be absorbed by a rubber fender during compression, as illustrated by curve 1 in Fig. 14.19, is partially restored to the mass which acts on it during compression, as illustrated by curves 2 and 3, and partially dissipated in the form of heat within the rubber material itself. This latter effect is called the hysteresis effect, and is represented by the area between the curves 1 and 2 or 1 and 3. The ratio between the dissipated energy and the received energy will vary according to the type of rubber fender and will be in the order of about 0.1 to 0.4.

The hysteresis effect can also be illustrated by dropping a rubber ball to the floor. If the ball acts as a 'bounce' ball, the hysteresis effect is very small and a fender of this material will be a recoiling fender. If a ship therefore hits a fender of this type of rubber material, the ship will be thrown out from the fender after the fender has absorbed the berthing energy. On the other hand, if the ball acts as a dead ball, the hysteresis effect is very large; a fender of this material will be a non-recoiling fender, and a ship will not be thrown out from the fender. In this case the ship will very slowly be pushed out from the fender. Therefore a fender with a large hysteresis effect will have the best effect on the mooring of a ship and the reduction of sway movements.

In Fig. 14.20 the principle of a permanent moored floating structure is illustrated without the use of ordinary mooring lines. To reduce the ship movement as much as possible this can be done as illustrated by

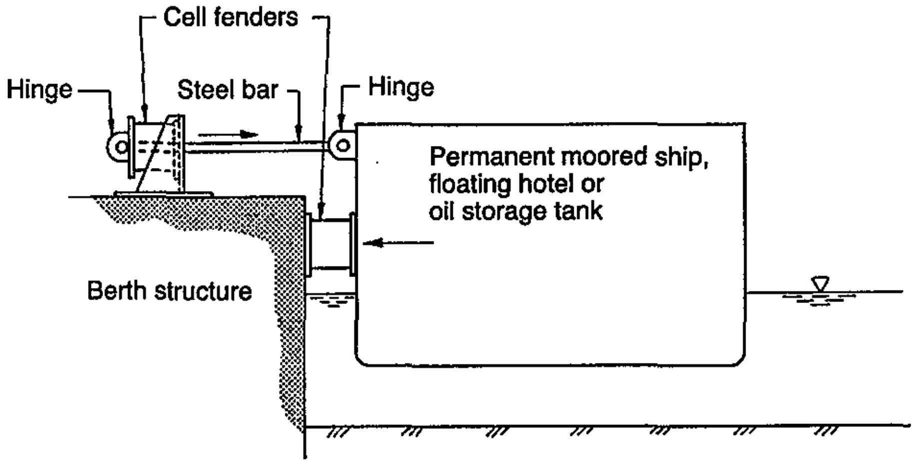


Fig. 14.20. Permanent fender mooring

Bridgestone, Japan with cell fenders with a high hysteresis effect acting against each other.

14.7.4 Temperature

The influence of temperature on a cell fender is illustrated, in principle, in Fig. 14.21. As the temperature becomes lower the reaction force will rise, and for a rise in temperature the reaction force will decrease. For temperatures lower than about −35 to −40 °C, the rubber compound should be specially designed, because the brittle point of the rubber is

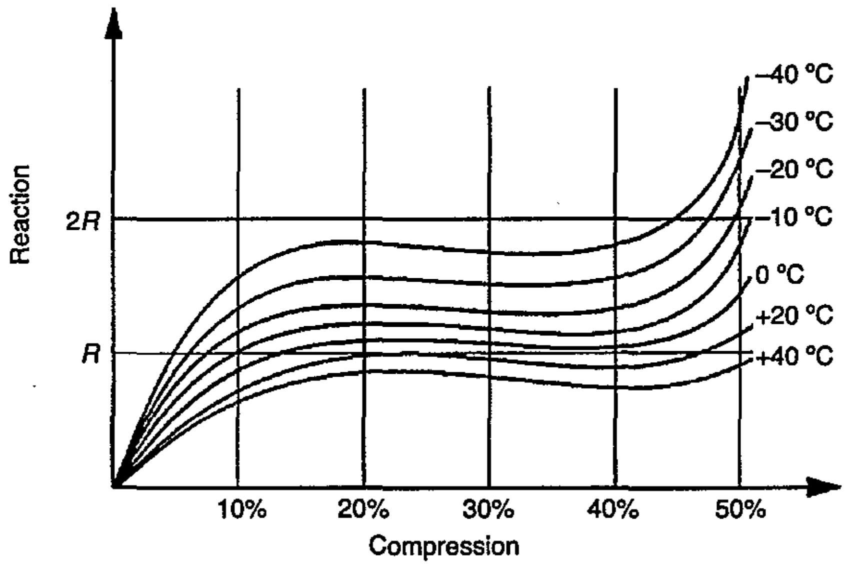


Fig. 14.21. The compressive performance of a cell fender under different temperatures

about −55 °C. Therefore for example some fender manufacturers have developed a special rubber type for arctic conditions.

14.7.5 Friction

The friction coefficient between the ship and the fender itself will depend upon the surface materials of the fender. To prevent damage to the fender due to the forward and/or backwards movements of the ship, the friction coefficient should be as low as possible. The following friction coefficient can be assumed:

(a) steel to special low friction materials 0.1–0.2
(b) steel to steel 0.2–0.3
(c) steel to timber 0.4–0.6
(d) steel to rubber 0.6–0.7.

Generally during tension or forced mooring one may need high lateral or horizontal resistance against surge movements due to long periodic waves like seiches acting along the berth front, and a low vertical resistance against heave, roll and pitch. Fender walls covered with hardwood, like azobe, have a horizontal friction factor of only about 0.3 and for vertical movement about 0.2. On the other hand, fender walls with small cylindrical rubber rollers with a horizontal axis can provide a horizontal friction coefficient of about 0.6–0.7, but only a vertical friction coefficient of about 0.1.

Since most fender systems are weak against forces acting parallel to the berthing face from berthing and moored ships, the fender wall must be designed for these parallel forces, which can act both vertically and longitudinally. The most common way to prevent failure of the fender wall and the fender is to anchor the fender wall by chains in such a way as to limit the vertical and longitudinal motion and to use low friction plastic pads on the fender walls.

14.8 Single- and double-fender systems

In the single-fender system one layer of fenders is mounted on the berth front. Examples are shown in Fig. 14.22(a) and (b). In the double-fender system two layers of fenders are used, one outside the other, with a plate or wall in between, as shown in Fig. 14.22(c) and (d).

The advantage offered by the double-fender system, shown in Fig. 14.22(c), is that it can absorb twice as much impact energy for the same reaction forces. However, the impact force on the berth

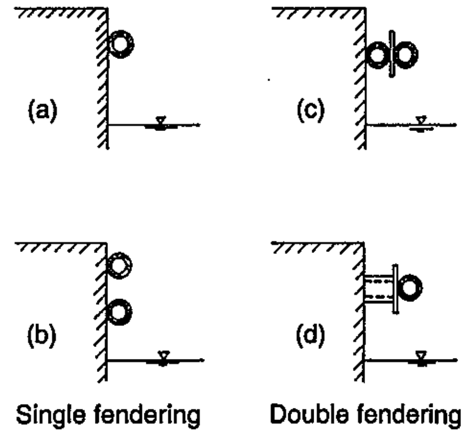


Fig. 14.22. Single- and double-fender systems

structure will remain the same with both systems. In other words, the fender factor will be halved when using this double-fender system. Under normal conditions, the impact loads during berthing are small, but upon maximum impact caused by a ship striking a berth structure these double fenders are required to absorb abnormally high energy without causing damage to either ship or berth.

The double-fender system shown in Fig. 14.22(c)–(d), is often called an ideal fender system because of its energy absorption and reaction force characteristics. When a cell fender and a cylindrical fender are combined in a double-fender system as illustrated in Fig. 14.23, the cylindrical fender will 'soften' the reaction/compression characteristics of the double-fender unit. This will make the double-fender more useful and it will act as an energy-absorbing fender also for smaller ships. The cylindrical fender must be so large that the reaction force

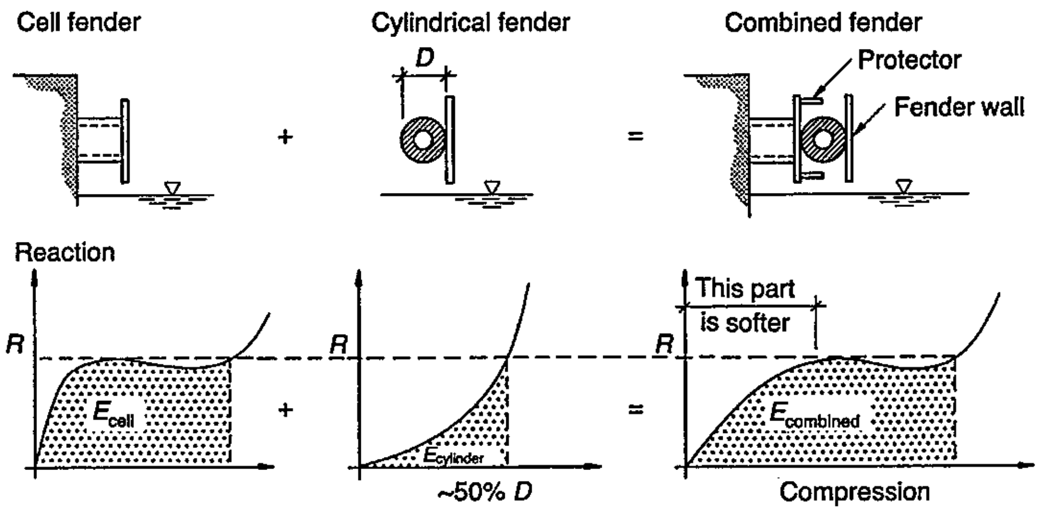


Fig. 14.23. Reaction/compression characteristics of double fender

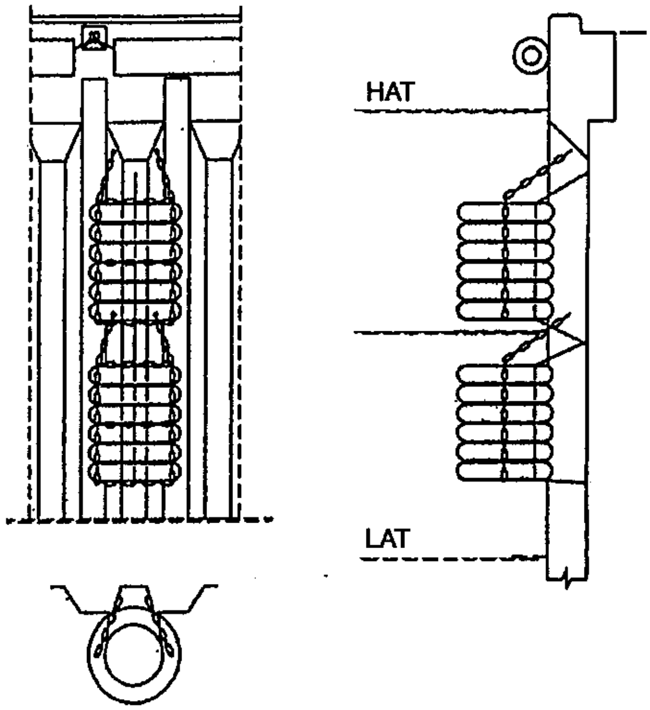


Fig. 14.24. RTT fender system

when the cylindrical fender is closed and the compression is equal to about 50 per cent of the outside diameter is equal to the reaction force needed to compress the cell fender. In order to prevent the cylindrical fender from being compressed by more than about 50 per cent, a compression stopper or protector can be mounted, as illustrated in Fig. 14.23.

The design of a double-fender system should be arrived at according to the trial-and-error method, and the procedure will be as follows:

(a) calculate the ship's impact energy
(b) choose an impact force *P* equal to the horizontal force that the berth structure or ship's hull can resist divided by a safety factor
(c) check that the total fender energy absorbed is at least equivalent to the ship's impact energy
(d) check the fender factor.

It is very difficult to design a fender system that can be a soft fender system for smaller ships and at the same time is a good energy-absorbing fender system for larger ships. The Port of Reykjavik, Iceland, has developed a fender system called the Reykjavik Truck Tyre fender system, or the RTT fender system, as shown in principle in Fig. 14.24. It consists of 6 connected truck tyres with an outside diameter of about 1.1 m in a stack suspended on the outside berth structure constructed of steel sheet piles. On the top concrete cap beam energy absorbing fenders are mounted, as indicated in Fig. 14.24, using cylindrical fenders.

The RTT system is a combination of buckling fenders at the top of the berth structure, and tyre fenders hanging along the lower front sheet pile wall with their fender line a little outside the fender line for the buckling fender. In this system one has a fender system with good fender properties for the smaller ship and at the same time soft enough for the larger ship to compress the tyre fenders before it berths against the buckling fender. The system has been suitable for ships in the range between 4500 and 60 000 dwt.

With the concrete cap beam about 0.3 m outside the front of the steel sheet piles, while using a cylindrical or buckling fender protruding about 0.4–0.5 m outside the concrete berth line, the buckling fender will act together with the tyre fenders when the tyres have been compressed about 40–50 per cent.

14.9 Fender wall

It is customary to berth the ship directly to the fender itself, as illustrated in Fig. 14.25(a), whether it is a single- or double-fender system. The fender here is suspended from the berth front and the ships will have direct contact with the fender. Chains suspend the ordinary cylindrical rubber fenders, whereas the larger cylindrical fenders are suspended by a bracket or ladder system. While absorbing the berthing energy of a ship the fender will give a reaction force to both the ship and the berth structure. Under normal berthing conditions no plastic deformation of the ship's hull should take place.

To prevent excessive concentration of the ship mooring forces, as well as berthing forces, both to the fendering systems itself and to the ship's hull, a protection panel or wall should be provided as required to reduce the face pressure.

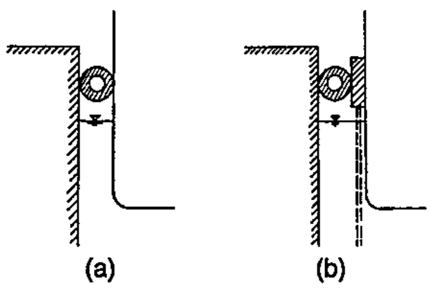


Fig. 14.25. Berthing directly to fender or fender plate

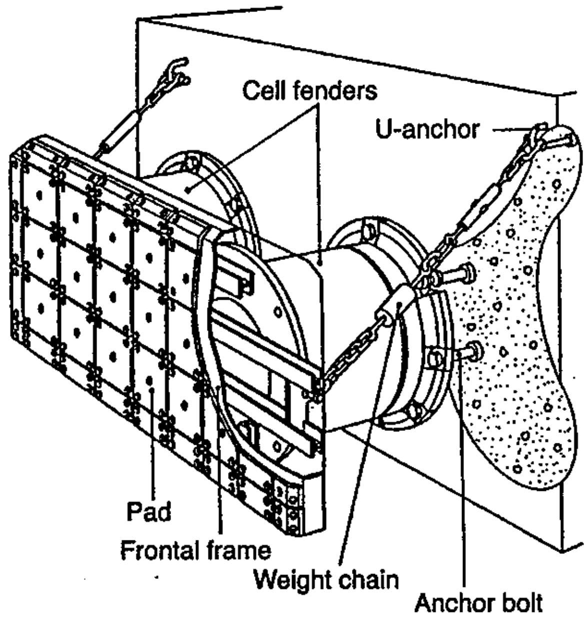


Fig. 14.26. The Bridgestone cell fenders

If there are large differences in tidal range, or in the ship's waterline (loaded or in ballast), or if the ship's side requires a greater contact surface against the fender structure to restrict the hull-bearing pressure on the ship, a fender wall is usually placed between the fenders and the ship's hull, as illustrated in Fig. 14.25(a)–(b). This fender-wall method can also be applied when minimum or maximum friction between ship and berth structure is required. Fender walls, made of steel, azobe or greenheart, with a rubber fender behind, have proved to be economic as well as effective. When tankers berth, the fender system will have to absorb energy from about 1000 kN m (small tankers) up to 6000 kN m (larger tankers). To obtain the dimensions of the fender wall one divides the impact force by the permitted ship hull load/m^2. Figure 14.26 shows, in principle, how two cell fenders are mounted behind a fender wall, and Fig. 14.27 displays cell fenders with fender walls.

Below, different fender systems with a fender wall in front of the fender are shown. In principle there are the following two wall systems:

(a) The fender wall can rotate in front of the fender, as shown in Fig. 14.27.
(b) The fender wall moves parallel sidewise regardless of where the flare angle or the ship fender belting is hitting the fender wall, as shown in Figs 14.28, 14.29 and 14.30.

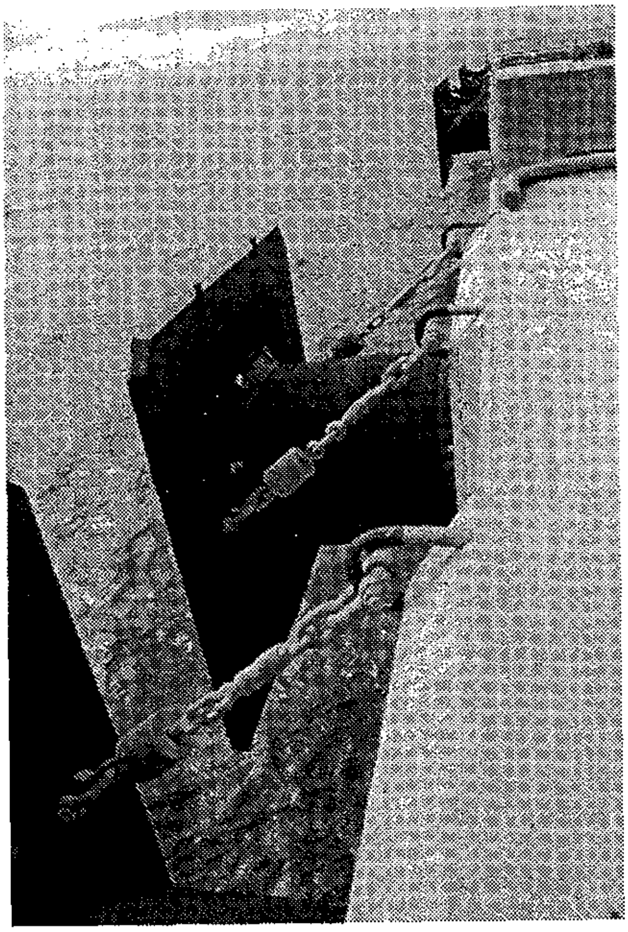


Fig. 14.27. Bridgestone cell fenders with fender walls

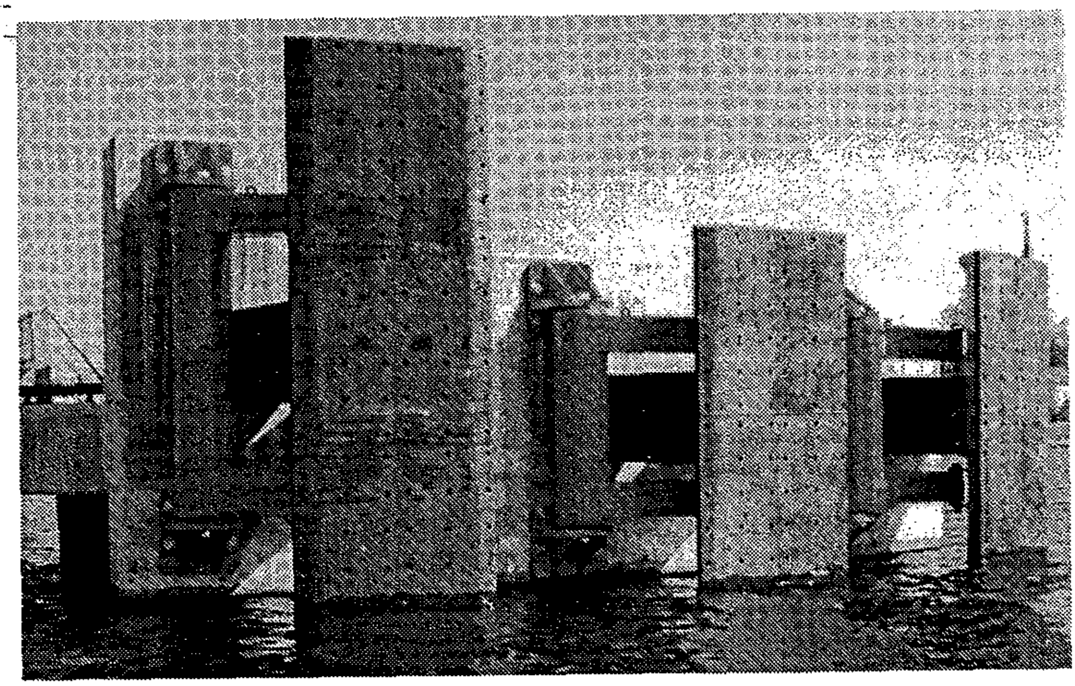


Fig. 14.28. Trellex parallel fender system

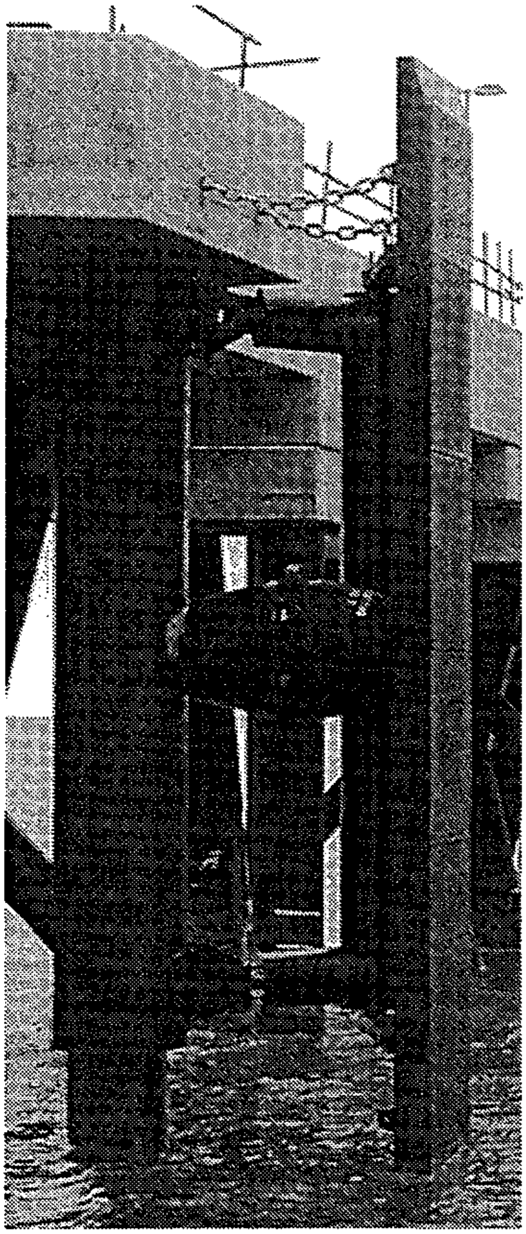


Fig. 14.29. Fentek parallel fender wall system with cone fenders

14.10 Hull pressure

Due to the variety in both the design and the type of ships, there are no firm or exact values for the allowable hull-bearing pressure which can be associated with the different type or size of ship, but as a very rough guideline the following permissible hull pressures can be used:

Type of ship	Hull pressure kN/m^2
General cargo	
<20 000 ton displacement	400–700
>20 000 ton displacement	<400

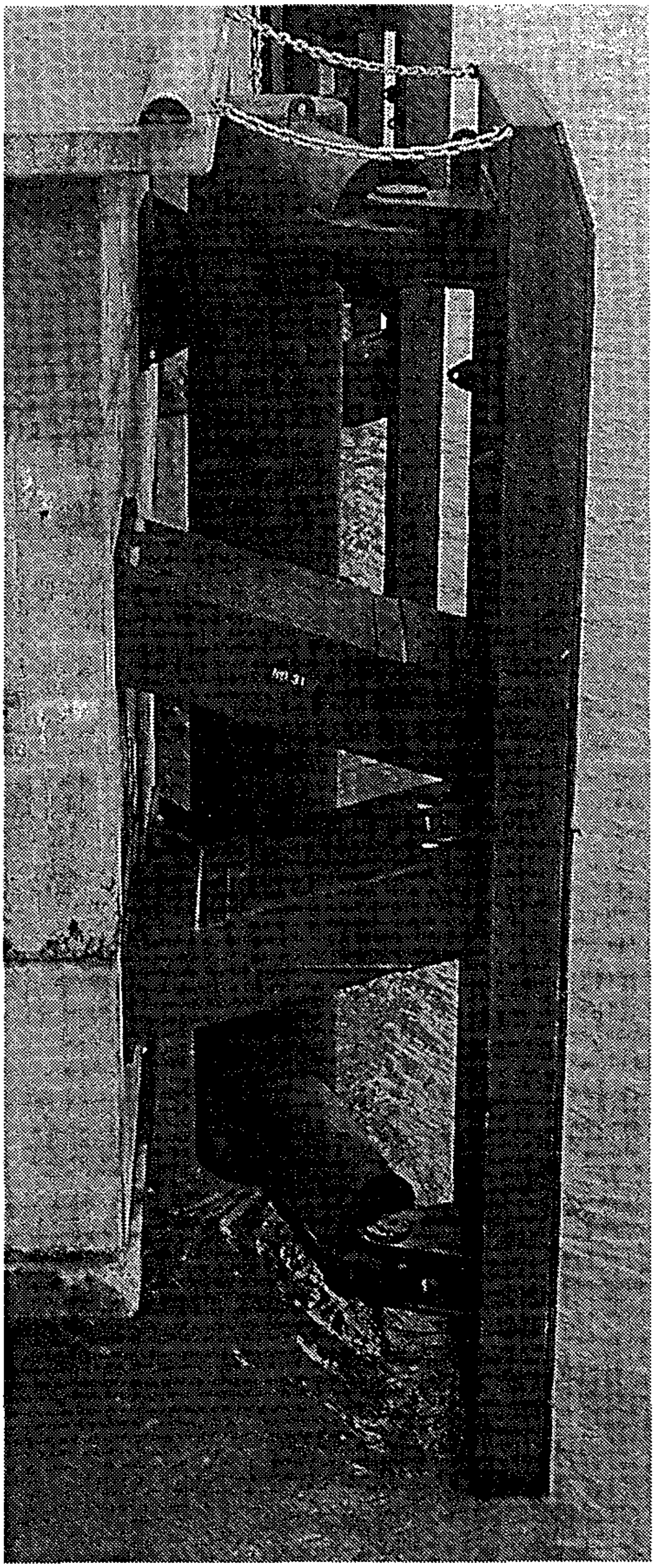


Fig. 14.30. Fentek parallel wall system with unit element fenders

Container ships	
1st and 2nd generation	300–500
3rd generation	<300
4th generation	<250
5th and 6th generation	<200
Oil tankers	
<60 000 ton displacement	250–350
>60 000 ton displacement	<350
VLCC	150–200
Bulk carriers	<200
Gas tankers (LNG/LPG)	<200

Ships with belting produce a line load on the fenders that can be considerably higher than the hull pressure indicated above.

The hull resistance to impact must be at least equal to the maximum hydrostatic pressure, which can act on the ship's hull.

Special attention should be paid to the horizontal chains on a fender panel. When chains are installed below the fender, the rotation of the fender panel, due to the ship's flare, can be restricted. Line loads may occur which exceed the permissible hull pressure.

14.11 Spacing of fenders

The spacing of fenders varies from berth structure to berth structure, depending on the type of structure, the requirements to be met by the berth and the type of ships using the berth. For ships not using tugboat assistance during berthing, the fender spacing will, in most cases, be determined by the smallest ship using the berth and by the ship hull radius of curvature, as illustrated in Fig. 14.31. The fender

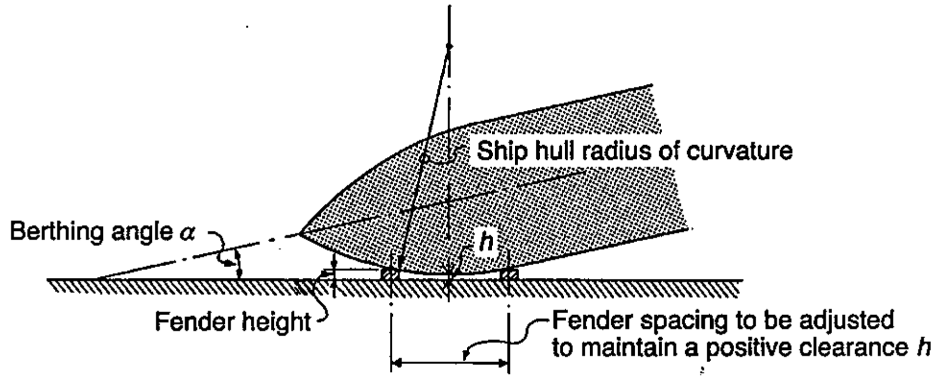


Fig. 14.31. Spacing of fenders

spacing will also depend on the fender height and the compression of the fenders.

Generally, to ensure that all ships can be supported at the berth, the fender spacing will be about 5–10 per cent of the ship's length for ships up to about 20 000 DWT. For larger ships the spacing can be about 25–50 per cent of the ship's length, if the ship berths with tugboat assistance. For optimum effect, the fenders for larger ships should be located close to the ends of the straight-sided section of the ship.

14.12 Cost of fenders

Figure 14.32 gives an indication of the relative price/lin m of cylindrical rubber fenders for a single-fender system with respect to fender factor.

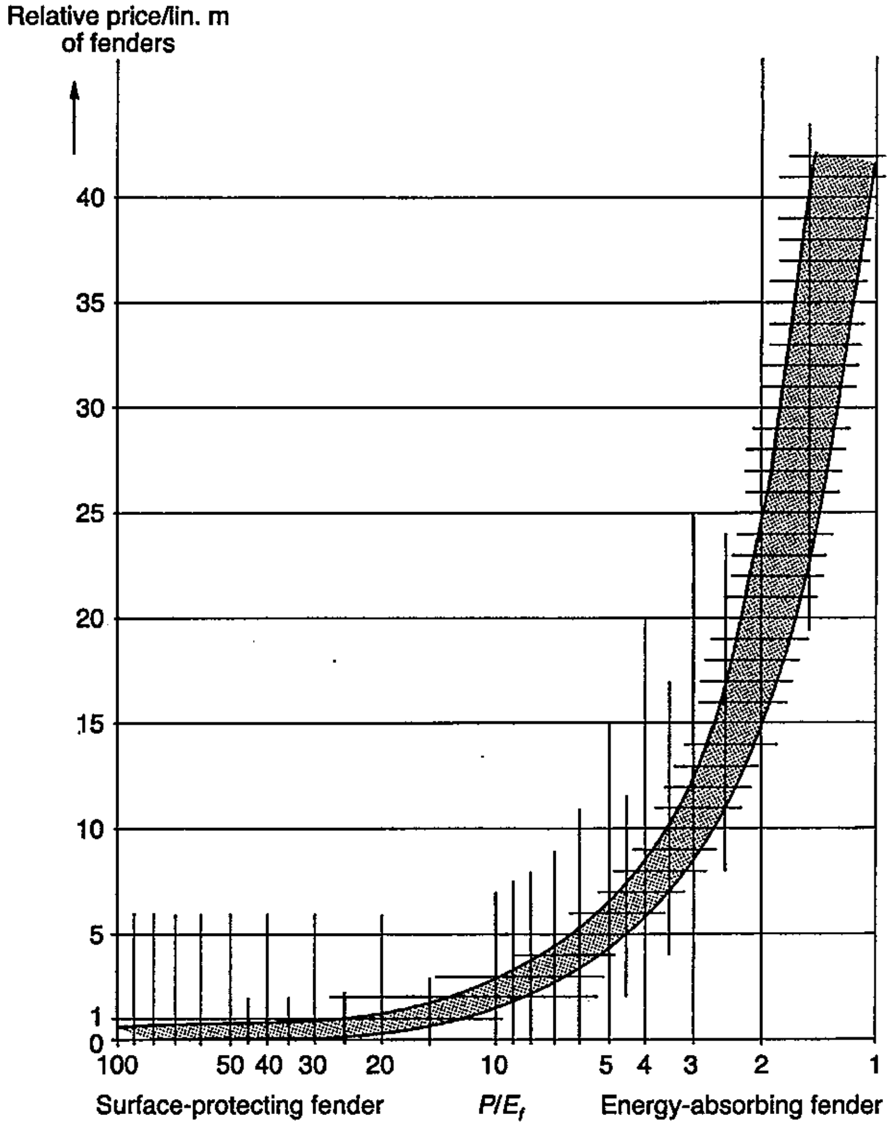


Fig. 14.32. Relative price of cylindrical rubber fenders

As can be seen, the relative price remains approximately constant for surface-protecting fenders but increases sharply according to energy-absorbing demands. The cost of the fenders should take into account the frequency of the berthing operations. A high frequency of berthing will normally justify a greater capital expenditure for the fender system.

One should realize that if energy-absorbing fenders with a fender factor of 3 or lower are required it could be more advantageous to install a double-fender system instead of a single-fender system. This is in spite of the fact that a double-fender system involves higher maintenance costs. For instance, a single-fender system, as shown in Fig. 14.22(a), with fender factor 2 will have a relative price/lin m of about 25, whereas two single-fender systems, each with fender factor 4, mounted as a double-fender system, as shown in Fig. 14.22(c), which will give the fender system a fender factor 2, will have a relative price of about $2 \times 10 = 20$.

14.13 Damage to fender structures

In general, damage to fender structures can be divided into two groups:

(a) Damage to be paid by the port owner, e.g. ordinary wear and tear by ships, or consequences suffered through incorrect type of fender, faulty mounting, etc.
(b) Damage to be paid by the ship owner due to crashing into berth structure during berthing, damage caused by ships' steel fenders, etc.

Apart from ordinary wear and tear, a port owner should be spared the damage mentioned under item (a). Of those mentioned under item (b), the damage caused by ships' steel fenders will cause annoyance and create extra work. Even if insurance companies compensate for damage inflicted, no payment is provided for the work involved in obtaining the compensation. More often, it pays to invest more money in the construction or upgrading of berth structures so that ships with steel fenders do not cause damage apart from ordinary wear and tear.

It has often been suggested that the use of steel fenders or belting should be prohibited. However, it would be better to urge ship owners to invest in steel fenders of such types that do less harm to the berth structures. Ship owners are undoubtedly right in stating that steel fenders protect the ship's sides when entering sluices, docks, etc.

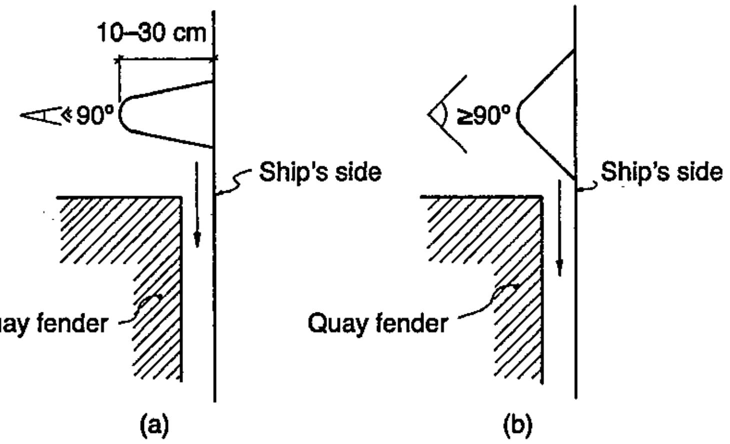


Figure 14.33 detail (a) shows the most common type of ship's steel fenders or belting. When the water level falls or while a ship takes on cargo, it may happen that a ship's fender gets stuck on top of the berth fender and that the ship will have difficulty in becoming detached. At worst, the berth fender may break. With the type shown in Fig. 14.33 detail (b), the ship will have no difficulty in sliding off the berth fender without damage to the latter. This type of ship's fender will also cause less chafing against wooden fender structures during loading and unloading.

Figure 14.34 illustrates three different types of fender as opposed to ship's steel fenders. Type (a) shows a pneumatic fender, (b) a hardwood fender wall and (c) a steel pile fender on the outside of the rubber fender. A wall made of hardwood has proved to be resistant to wear and tear, but in most cases a steel fender wall would be a better choice. The low maintenance cost of a fender wall often justifies its high prime cost.

When using a fender wall, the wall should always be constructed at the top or bottom, as shown in principle in Fig. 14.35.

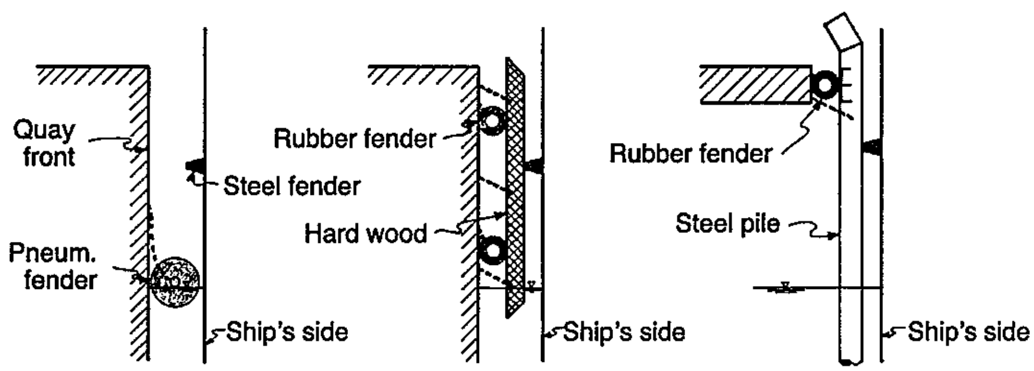


Fig. 14.34. Different ways of fendering the berth against ship's steel fender

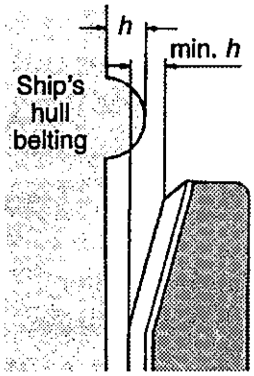


Fig. 14.35. Detail of fender wall

14.14 Calculation examples

The calculations are based on the theory described in Chapter 5.

Example 1

Figure 14.36 shows a cross-section of an open pier structure for ships of 6000 t displacement on the 95 per cent confidence level. The ship's length, width and depth are approximately 105 m, 14.2 m and 6.4 m respectively. One assumes that $\phi = 60°$ and that the hydrodynamic mass coefficient is only about 70 per cent because the ship does not move perpendicularly to its longitudinal axis. The ship's berthing velocity of approach is 0.25 m/s of the confidence level.

$$C_H = \frac{6000 + (1/4 \times \pi \times 1.03 \times 6.4^2 \times 105) \times 70\%}{6000} = 1.4$$

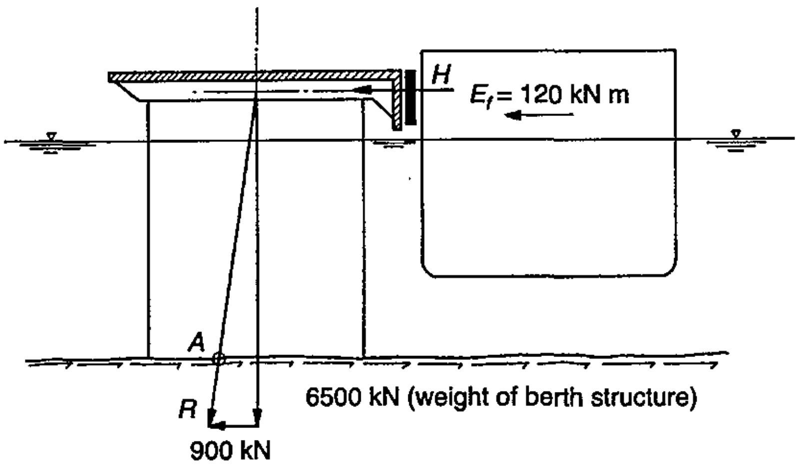


Fig. 14.36. Example: cross-section of open pier structure

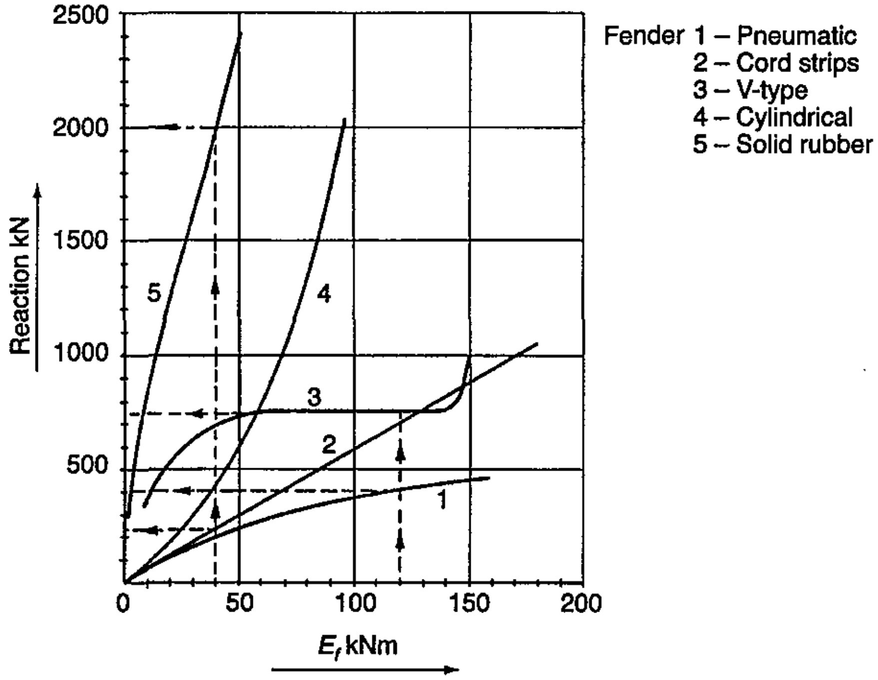


Fig. 14.37. Curves representing different energy absorption

One assumes that r (distance from the centre of gravity of the vessel to the point of contact) $= 0.3 \times 1$, i.e. $r/l = 0.3$, which gives $C_E = 0.48$. Assumes that $C_C = 1.0$ and $C_S = 0.95$ and the adjusting factor $C = C_H \times C_E \times C_C \times C_S = 0.64$.

The energy to be absorbed by the fender structure without an abnormal impact factor will be:

$$E_f = 0.64 \times (0.5 \times 6000 \times 0.25^2) = 120\,\text{kN m}$$

Out of the five different fenders represented in Fig. 14.37, fenders nos. 2, 4 and 5 absorb the impact energy over a fender length of 3 m, fender no. 1 and fender no. 3, over a length of 3.5 m and 2 m respectively. Curves 1 and 3 give the total reaction force whereas curves 2, 4 and 5 give the reaction force, in linear metres, which has to be multiplied by 3 m to obtain the total reaction force. This is shown in Fig. 14.38 where type, dimension, cross-section, load/lin m and total reaction force are given.

The pneumatic fender, no. 1, gives the lowest load factor. Because of its dimensions, it is best suited for oil harbours where the distance to berth front is of minor importance as ships are unloaded by means of pipes and loading arms.

The other fender types, nos. 2, 3 and 4, have the same criteria where general cargo berth structures are concerned. Here it is just a question

No.	Rubber fender (type and dimensions)	Cross-section	E_f and P noted	Loading (kN/lin m)	Total impact reaction (P kN)	P/E_f
1	Pneumatic $\phi = 2$ m $l = 3.5$ m	○	Total		400	3.33
2	Cord strips $\phi = 1$ m	○	Per lin m	230	690	5.75
3	V-type $h = 0.5$ m $1 = 2$ m	⏢	Total		750	6.25
4	Cylindrical $\phi = 0.61$ m	∘	Per lin m	400	1200	10
5	Solid $h = 0.15$ m	▬	Per lin m	2000	6000	50

Fig. 14.38. Fender factor of different types of rubber fender

of how much one is willing to, or has to pay, to absorb the impact energy on account of berth stability, etc.

Fender no. 5 is included to give a general idea of the various types of fenders. However, its purpose is not so much to absorb the energy as to protect both ships and berth structures against rubbing and damage.

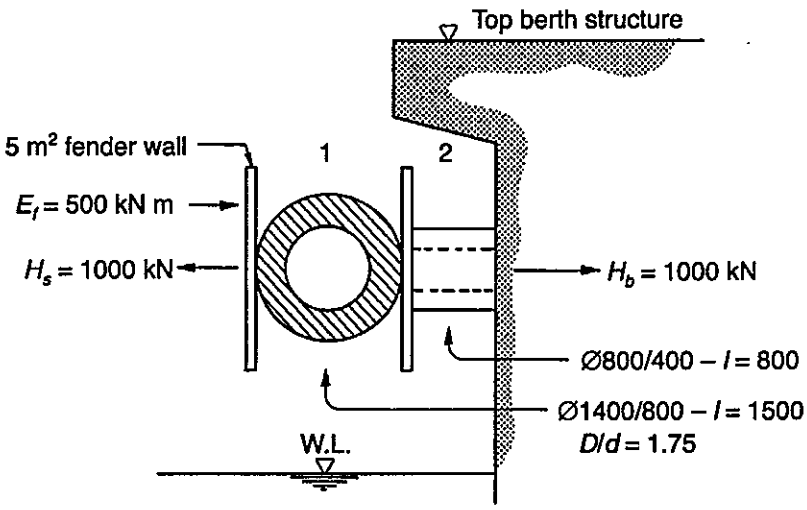
If the resultant R in Fig. 14.36 shall act at A, i.e. within the mid one-third of the width of the structure, the horizontal force should not exceed 900 kN. With a safety factor of 1.3, the dimensional horizontal force is $900/1.3 = 700$ kN, i.e. a fender with a lower fender factor than $P/E_f = 700/120 = 5.8$ kN/kN m must be found. According to the table in Fig. 14.38, fender nos. 1 and 2 with respective fender factors 3.33 and 5.75 kN/kN m will meet these requirements. Fender no. 4, mounted as a double-fender system will also meet these requirements.

Example 2

Figure 14.39 shows a calculation example of a double-fendering system where the berthing energy is 500 kN m and the horizontal force on the shipside or the berth structure should not exceed 1000 kN, based on a 95 per cent confidence level for the displacement, a 50 per cent confidence level of the berthing velocity and a factor for abnormal impact of 1.5.

14.15 Information from different fender manufacturers

From the following different fender manufacturers, the performance values for their different fender types are given in the following sections.



	Fender 1	Fender 2	Σ (F1 + F2)	Fender factor
Horizontal load	1000 kN	1000 kN	1000 Kn	$P/\Sigma E_f = \sim 2.0$
Compression	77.5 cm	24.5 cm	102 cm	
Energy absorbed	350 kN m	135 kN m	485 kN m	

With fender wall

Surface pressure against fender wall $\frac{1000}{5} = 200$ kN/m^2

Without fender wall

Contact area of compressed fender 1 = $A = 0.9 \times D \times l = 1.89$ m^2

Surface pressure against ship side $\frac{1000}{1.89} = 530$ kN/m^2

Fig. 14.39. Calculation example of double-fender system

14.15.1 Fentek marine fendering system

Cylindrical fenders

The energy absorption and reaction forces for cylindrical fenders are shown in Table 14.1. The fender performances in the figure are for 1000 mm length and for a rated deflection equal to the inside diameter. For a stable installation the length L should be larger than the outside diameter D.

Super cone fenders

All energy absorption and reaction force values are at a rated deflection of 72 per cent. The maximum deflection of the fender is 75 per cent and where the compressive loads may exceed the maximum fender reaction

Table 14.1. Energy absorption and reaction forces on different sizes of 1000 mm long **cylindrical fenders**

D (mm)	d (mm)	R (kN)	E (kN m)	P (kN/m^2)	ε (E/R)	Wt (kg/m)
Extruded						
100	50	43	0.8	547	0.019	7.0
125	65	51	1.3	500	0.025	10.6
150	75	65	1.8	552	0.028	15.6
175	75	92	2.7	781	0.029	23.2
200	90	98	3.5	693	0.036	29.6
200	100	86	3.3	547	0.038	27.8
250	125	108	5.1	550	0.047	43.4
300	150	129	7.4	547	0.057	62.6
380	190	164	11.8	550	0.072	100.4
400	200	172	13.1	547	0.076	111.2
450	225	194	16.6	549	0.086	140.8
500	250	275	28	700	0.102	175
Wrapped						
600	300	330	40	700	0.121	253
700	400	325	52	517	0.160	309
750	400	380	61	605	0.161	377
800	400	440	72	700	0.164	449
875	500	406	81	517	0.200	482
925	500	461	93	587	0.202	567
1000	500	550	112	700	0.204	702
1050	600	487	117	517	0.240	695
1100	600	541	131	574	0.242	795
1200	600	660	162	700	0.245	1010
1200	700	542	151	493	0.279	889
1300	700	650	184	591	0.283	1122
1300	750	595	178	505	0.299	1055
1400	700	770	220	700	0.286	1375
1400	750	705	214	598	0.304	1307
1400	800	649	208	516	0.320	1235
1500	750	825	253	700	0.307	1579
1500	800	760	246	605	0.324	1506
1600	800	880	288	700	0.327	1796
1600	900	757	273	535	0.361	1637
1650	900	812	295	574	0.363	1789
1750	900	929	340	657	0.366	2107
1750	1000	811	325	516	0.401	1929
1800	900	990	364	700	0.368	2273
1850	1000	921	372	586	0.404	2266
2000	1000	1101	450	701	0.409	2806
2000	1200	871	415	462	0.476	2395
2100	1200	974	467	517	0.479	2778
2200	1200	1083	524	575	0.484	3180
2400	1200	1321	647	701	0.490	4041

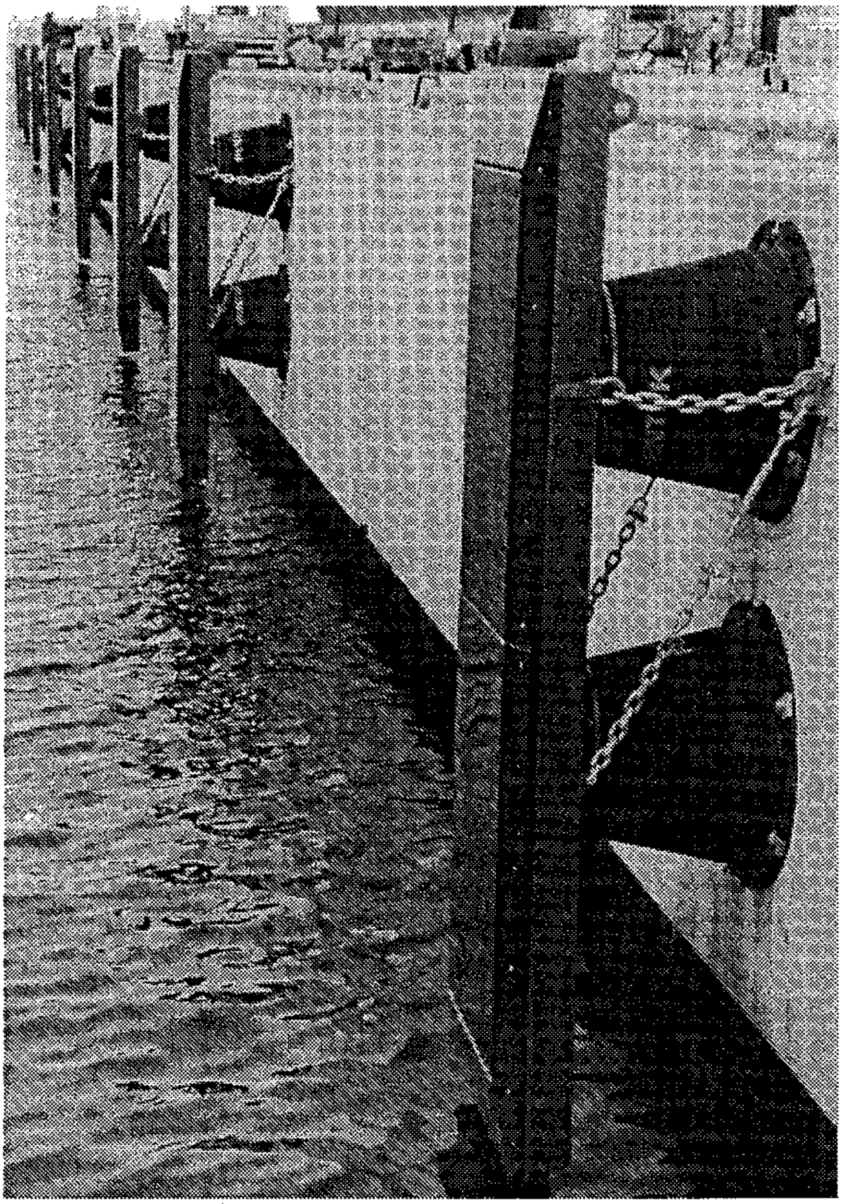


Fig. 14.40. Fentek super cone fender

an overload stopper may be used. The cone fenders are shown in Fig. 14.40 and the energy and reaction forces are shown in Table 14.2.

Unit element fenders

The unit element fenders are shown in Fig. 14.41 installed behind a fender wall.

All energy absorption and reaction force values are at a rated deflection of 57.5 per cent. The maximum deflection of the fender is 62.5 per cent. The performance values are for a single element 1000 mm long. The unit element fenders are shown in Table 14.3.

Table 14.2. Energy absorption and reaction forces on different sizes of **cone fenders**

		Super cone																	
Energy index	Fender size	SCN 300	SCN 350	SCN 400	SCN 500	SCN 550	SCN 600	SCN 700	SCN 800	SCN 900	SCN 1000	SCN 1050	SCN 1100	SCN 1200	SCN 1300	SCN 1400	SCN 1600	SCN 1800	SCN 2000
E0.9	*E* (kN m)	7.7	12.5	18.6	36.5	49	63	117	171	248	338	392	450	585	743	927	1382	1967	2700
	R (kN)	59	80	104	164	198	225	320	419	527	653	720	788	941	1103	1278	1670	2115	2610
E1.0	*E* (kN m)	8.6	13.9	20.7	40.5	54	70	130	190	275	375	435	500	650	825	1030	1535	2185	3000
	R (kN)	65	89	116	182	220	250	355	465	585	725	800	875	1045	1225	1420	1855	2350	2900
E1.1	*E* (kN m)	8.9	14.4	21.4	41.9	56	72	134	196	282	385	447	514	668	847	1058	1577	2244	3080
	R (kN)	67	91	119	187	226	257	365	478	601	745	822	899	1073	1258	1459	1905	2413	2978
E1.2	*E* (kN m)	9.2	14.8	22.1	43.2	58	74	137	201	289	395	458	527	685	869	1085	1618	2303	3160
	R (kN)	68	93	122	191	231	263	374	490	617	764	843	923	1101	1291	1497	1955	2476	3056
E1.3	*E* (kN m)	9.5	15.3	22.8	44.6	59	76	141	207	296	405	470	541	703	891	1113	1660	2362	3240
	R (kN)	70	96	125	196	237	270	384	503	633	784	865	947	1129	1324	1536	2005	2539	3134
E1.4	*E* (kN m)	9.8	15.7	23.5	45.9	61	78	144	212	303	415	481	554	720	913	1140	1701	2421	3320
	R (kN)	72	98	128	200	242	276	393	515	649	803	886	971	1157	1357	1574	2055	2602	3212
E1.5	*E* (kN m)	10.1	16.2	24.2	47.3	63	80	148	218	310	425	493	568	738	935	1168	1743	2480	3400
	R (kN)	74	100	131	205	248	283	403	528	665	823	908	995	1185	1390	1613	2105	2665	3290
E1.6	*E* (kN m)	10.4	16.7	24.8	48.6	65	82	151	223	317	435	504	581	755	957	1195	1784	2539	3480
	R (kN)	75	102	133	209	253	289	412	540	681	842	929	1019	1213	1423	1651	2155	2728	3368
E1.7	*E* (kN m)	10.6	17.1	25.5	50.0	67	84	155	229	324	445	516	595	773	979	1223	1826	2598	3560
	R (kN)	77	104	136	214	259	296	422	553	697	862	951	1043	1241	1456	1690	2205	2791	3446
E1.8	*E* (kN m)	10.9	17.6	26.2	51.3	68	86	158	234	331	455	527	608	790	1001	1250	1867	2657	3640
	R (kN)	79	107	139	218	264	302	431	565	713	881	972	1067	1269	1489	1728	2255	2854	3524
E1.9	*E* (kN m)	11.2	18.0	26.9	52.7	70	88	162	240	338	465	539	622	808	1023	1278	1909	2716	3720
	R (kN)	80	109	142	223	270	309	441	578	729	901	994	1091	1297	1522	1767	2305	2917	3602

E2.0	E (kN m)	11.5	18.5	27.6	54.0	72	90	165	245	345	475	550	635	825	1045	1305	1950	2775	3800
	R (kN)	82	111	145	227	275	315	450	590	745	920	1015	1115	1325	1555	1805	2355	2980	3680
E2.1	E (kN m)	11.8	19.0	28.3	55.4	74	93	169	252	355	488	565	652	847	1074	1341	2003	2851	3904
	R (kN)	84	114	149	233	283	324	462	606	765	945	1042	1145	1361	1597	1853	2418	3060	3778
E2.2	E (kN m)	12.1	19.4	29.0	56.7	76	96	173	258	364	501	580	669	869	1102	1376	2056	2926	4008
	R (kN)	86	117	153	239	290	332	474	621	785	969	1069	1174	1396	1638	1901	2480	3139	3876
E2.3	E (kN m)	12.4	19.9	29.7	58.1	77	99	177	265	374	514	595	686	891	1131	1412	2109	3002	4112
	R (kN)	89	120	157	246	298	341	486	637	805	994	1096	1204	1432	1680	1949	2543	3219	3974
E2.4	E (kN m)	12.7	20.3	30.4	59.4	79	102	181	271	383	527	610	703	913	1159	1447	2162	3077	4216
	R (kN)	91	123	161	252	305	349	498	652	825	1018	1123	1233	1467	1721	1997	2605	3298	4072
E2.5	E (kN m)	13.0	20.8	31.1	60.8	81	105	185	278	393	540	625	720	935	1188	1483	2215	3153	4320
	R (kN)	93	126	165	258	313	358	510	668	845	1043	1150	1263	1503	1763	2045	2668	3378	4170
E2.6	E (kN m)	13.3	21.3	31.8	62.2	83	108	189	284	402	553	640	737	957	1216	1518	2268	3228	4424
	R (kN)	95	129	169	264	320	366	522	683	865	1067	1177	1292	1538	1804	2093	2730	3457	4268
E2.7	E (kN m)	13.5	21.7	32.5	63.5	85	111	193	291	412	566	655	754	979	1245	1554	2321	3304	4528
	R (kN)	97	132	173	270	328	375	534	699	885	1092	1204	1322	1574	1846	2141	2793	3537	4366
E2.8	E (kN m)	13.8	22.2	33.2	64.9	86	114	197	297	421	579	670	771	1001	1273	1589	2374	3379	4632
	R (kN)	100	135	177	277	335	383	546	714	905	1116	1231	1351	1609	1887	2189	2855	3616	4464
E2.9	E (kN m)	14.1	22.6	33.9	66.2	88	117	201	304	431	592	685	788	1023	1302	1625	2427	3455	4736
	R (kN)	102	138	181	283	343	392	558	730	925	1141	1258	1381	1645	1929	2237	2918	3696	4562
E3.0	E (kN m)	14.4	23.1	34.6	67.6	90	120	205	310	440	605	700	805	1045	1330	1660	2480	3530	4840
	R (kN)	104	141	185	289	350	400	570	745	945	1165	1285	1410	1680	1970	2285	2980	3775	4660
E3.1	E (kN m)	15.9	25.4	38.1	74.4	99	132	226	341	484	666	770	886	1150	1463	1826	2728	3883	5324
	R (kN)	114	155	204	318	385	440	627	820	1040	1282	1414	1551	1848	2167	2514	3278	4153	5126
Efficiency ratio (ε)		0.138	0.163	0.186	0.232	0.256	0.290	0.364	0.414	0.466	0.518	0.544	0.571	0.622	0.674	0.725	0.830	0.932	1.036

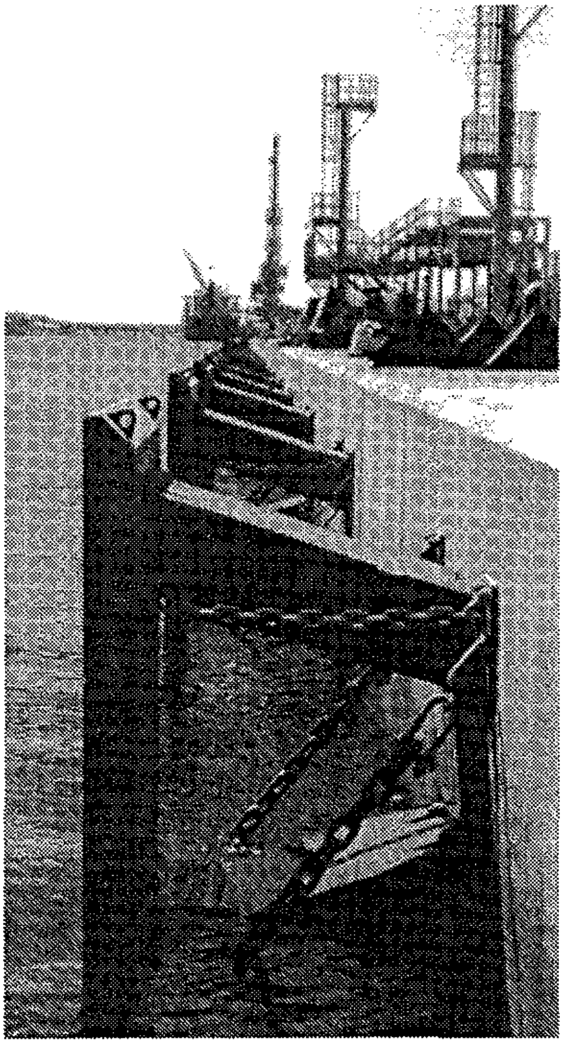


Fig. 14.41. The unit element fender

Arch fenders

All energy absorption and reaction forces values are at a rated deflection of 51.5 per cent for the AN fender and 54 per cent for the ANP fender. The arch fenders are shown in Fig. 14.42 and the energy and reaction forces are shown in Table 14.4.

14.15.2 Trellex fender system

MV and MI element fenders

The principle of the Trellex MV and MI element fender system is shown in Figs 14.43, 14.44 and 14.45.

The rated performances for one element of the MV and MI element fenders are shown in Tables 14.5 and 14.6.

Table 14.3. Energy absorption and reaction forces on different sizes of unit 1000-mm long element fenders

		Unit element													
Energy index	Fender size	UE 250	UE 300	UE 400	UE 500	UE 550	UE 600	UE 700	UE 750	UE 800	UE 900	UE 1000	UE 1200	UE 1400	UE 1600
E0.9	*E* (kN m)	8.1	11.7	21	32.4	40	47	63	73	84	106	131	186	257	337
	R (kN)	79	95	113	142	157	171	199	214	228	256	284	340	398	455
E1.0	*E* (kN m)	9.0	13.0	23	36.0	44	52	70	81	93	118	146	207	286	374
	R (kN)	88	105	126	158	174	190	221	238	253	284	316	378	442	506
E1.1	*E* (kN m)	9.3	13.4	24	37.1	45	54	72	84	96	122	150	213	294	385
	R (kN)	90	108	130	163	179	196	228	245	261	293	326	389	455	521
E1.2	*E* (kN m)	9.6	13.8	24	38.2	47	55	74	86	99	125	155	220	303	396
	R (kN)	93	111	134	167	184	201	234	252	268	301	335	401	469	536
E1.3	*E* (kN m)	9.9	14.2	25	39.3	48	57	77	89	101	129	159	226	311	407
	R (kN)	95	114	137	172	190	207	241	259	276	310	345	412	482	552
E1.4	*E* (kN m)	10.2	14.6	26	40.4	49	58	79	91	104	132	163	232	320	418
	R (kN)	98	117	141	177	195	212	247	266	283	318	354	424	495	567
E1.5	*E* (kN m)	10.5	15.0	27	41.5	51	60	81	94	107	136	168	239	328	429
	R (kN)	100	121	145	182	200	218	254	274	291	327	364	435	509	582
E1.6	*E* (kN m)	10.8	15.4	27	42.6	52	62	83	96	110	139	172	245	336	440
	R (kN)	103	124	149	186	205	224	261	281	299	336	373	446	522	597
E1.7	*E* (kN m)	11.1	15.8	28	43.7	53	63	85	99	113	143	176	251	345	451
	R (kN)	106	127	153	191	210	229	267	288	306	344	383	458	535	612
E1.8	*E* (kN m)	11.4	16.2	29	44.8	54	65	88	101	115	146	180	257	353	462
	R (kN)	108	130	156	196	216	235	274	295	314	353	392	469	548	628
E1.9	*E* (kN m)	11.7	16.6	29	45.9	56	66	90	104	118	150	185	264	362	473
	R (kN)	111	133	160	200	221	240	280	302	321	361	402	481	562	643

Table 14.3. Continued

		Unit element													
E2.0	*E* (kN m)	12.0	17.0	30	47.0	57	68	92	106	121	153	189	270	370	484
	R (kN)	113	136	164	205	226	246	287	309	329	370	411	492	575	658
E2.1	*E* (kN m)	12.3	17.5	31	48.5	59	70	95	109	125	158	195	278	381	499
	R (kN)	117	140	169	211	233	253	296	318	339	381	423	507	592	678
E2.2	*E* (kN m)	12.6	18.0	32	50.0	61	72	98	112	128	162	200	286	392	513
	R (kN)	120	144	174	217	240	261	305	328	349	392	436	522	610	697
E2.3	*E* (kN m)	12.9	18.5	33	51.5	62	74	100	115	132	167	206	294	404	528
	R (kN)	124	149	179	224	246	268	313	337	358	403	448	537	627	717
E2.4	*E* (kN m)	13.2	19.0	34	53.0	64	76	103	118	135	171	212	302	415	542
	R (kN)	127	153	184	230	253	276	322	347	368	414	460	552	644	736
E2.5	*E* (kN m)	13.5	19.5	35	54.5	66	79	106	122	139	176	218	311	426	557
	R (kN)	131	157	189	236	260	283	331	356	378	426	473	567	662	756
E2.6	*E* (kN m)	13.8	20.0	35	56.0	68	81	109	125	143	181	223	319	437	572
	R (kN)	134	161	194	242	267	290	340	365	388	437	485	582	679	776
E2.7	*E* (kN m)	14.1	20.5	36	57.5	70	83	112	128	146	185	229	327	448	586
	R (kN)	138	165	199	248	274	298	349	375	398	448	497	597	696	795
E2.8	*E* (kN m)	14.4	21.0	37	59.0	71	85	114	131	150	190	235	335	460	601
	R (kN)	141	170	204	255	280	305	357	384	407	459	509	612	713	815
E2.9	*E* (kN m)	14.7	21.5	38	60.5	73	87	117	134	153	194	240	343	471	615
	R (kN)	145	174	209	261	287	313	366	394	417	470	522	627	731	834
E3.0	*E* (kN m)	15.0	22.0	39	62.0	75	89	120	137	157	199	246	351	482	630
	R (kN)	148	178	214	267	294	320	375	403	427	481	534	642	748	854
E3.1	*E* (kN m)	16.5	24.2	43	68.2	83	98	132	151	173	219	271	386	530	693
	R (kN)	163	196	235	294	323	352	413	443	470	529	587	706	823	939
Efficiency ratio (ε)		0.103	0.124	0.183	0.230	0.254	0.276	0.319	0.341	0.368	0.414	0.461	0.548	0.645	0.737

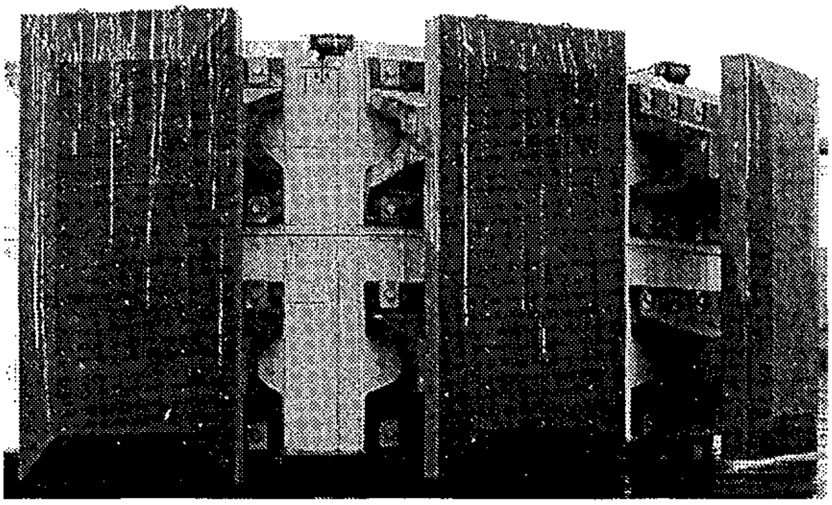


Fig. 14.42. Arch fender

Table 14.4. Energy absorption and reaction forces on different sizes of AN and ANP fenders

AN arch fender

AN	E1		E2		E3		Efficiency ratio (*e*)
	E (kN m)	*R* (kN)	E (kN m)	*R* (kN)	E (kN m)	*R* (kN)	
150	4.3	74	5.6	96.2	7.4	127	0.058
200	7.6	98.6	10	128	13.1	169	0.078
250	11.9	123	15.6	160	20.5	211	0.097
300	17.1	148	22.5	192	29.5	253	0.117
400	30.5	197	40	256	52.5	338	0.155
500	47.6	247	62.4	321	82	422	0.194
600	68.6	296	89.9	385	116	507	0.231
800	122	394	160	513	210	675	0.311
1000	191	493	250	641	328	844	0.389

ANP arch fender

ANP	E1		E2		E3		Efficiency ratio (*e*)
	E (kN m)	*R* (kN)	E (kN m)	*R* (kN)	E (kN m)	*R* (kN)	
150	5.6	88.8	7.3	115	9.5	150	0.063
200	9.9	118	12.9	154	16.8	200	0.084
250	15.6	148	20.2	192	26.3	250	0.105
300	22.4	178	29.1	231	37.8	300	0.126
400	39.8	237	51.7	308	67.2	400	0.168
500	62.1	296	80.8	385	105	500	0.210
600	89.3	355	116	462	151	600	0.251
800	159	473	207	615	269	800	0.336
1000	249	592	323	769	420	1000	0.420

Note: Performance values are for fender 1000 mm long

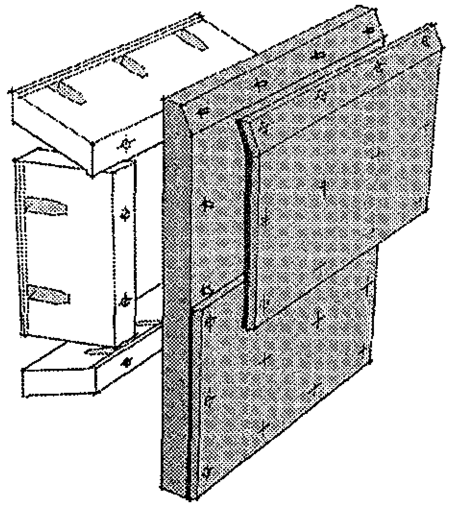


Fig. 14.43. Different construction of the Trellex MV and MI elements

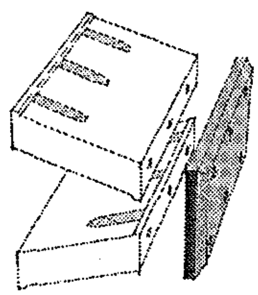


Fig. 14.44. Different construction of the Trellex MV and MI elements

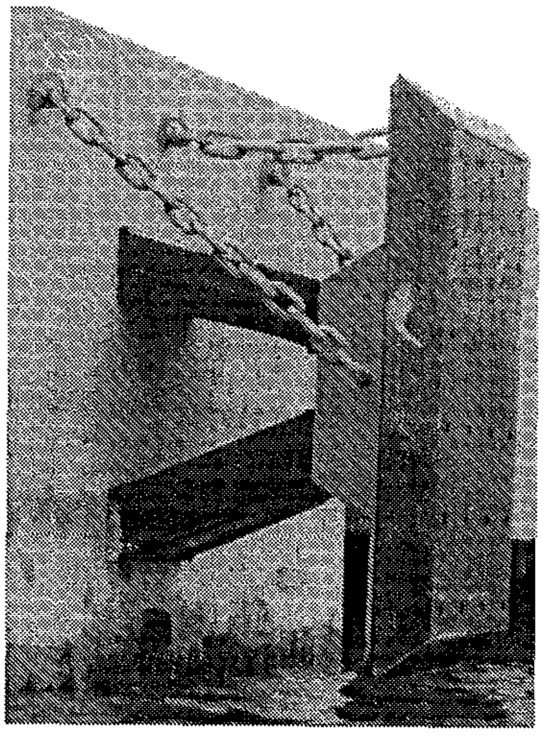


Fig. 14.45. Trellex element fender system

Table 14.5. The rated performance for one MV fender element

Element size H × L Compound A or B	Rated performance for one element					
	E (tonne/m)	*R* (tonne)	*E* (kN m)	*R* (kN)	*E* (ft/kips)	*R* (kips)
MV 250 × 890 B	1.0	8.5	9.8	83.4	7.2	18.7
MV 250 × 890 A	1.4	12.1	13.7	118.7	10.1	26.7
MV 250 × 1000 B	1.1	9.5	10.8	93.2	8.0	20.9
MV 250 × 1000 A	1.6	13.6	15.7	133.4	11.6	30.0
MV 300 × 600 B	0.9	6.8	9	66	6	15
MV 300 × 600 A	1.3	9.8	13	96	9	21
MV 300 × 900 B	1.4	10.3	14	101	10	22
MV 300 × 900 A	2.0	14.7	20	144	14	32
MV 300 × 1200 B	1.8	13.7	18	134	13	30
MV 300 × 1200 A	2.6	19.6	26	192	19	43
MV 300 × 1500 B	2.3	17.2	22	168	16	38
MV 300 × 1500 A	3.3	24.5	32	240	24	54
MV 400 × 1000 B	2.8	15.3	27	150	20	34
MV 400 × 1000 A	4.0	21.8	39	214	29	48
MV 400 × 1500 B	4.2	22.9	41	224	30	50
MV 400 × 1500 A	6.0	32.7	59	321	43	72
MV 400 × 2000 B	5.6	30.6	55	300	41	67
MV 400 × 2000 A	8.0	43.6	78	428	58	96
MV 400 × 2500 B	7.0	38.2	68	375	51	84
MV 400 × 2500 A	10.0	54.5	98	535	72	120
MV 400 × 3000 B	8.4	45.8	83	449	61	101
MV 400 × 3000 A	12.0	65.4	117	642	87	144
MV 500 × 1000 B	4.3	19.0	43	187	32	42
MV 500 × 1000 A	6.2	27.2	61	267	45	60
MV 500 × 1500 B	6.5	28.6	64	280	47	63
MV 500 × 1500 A	9.3	40.8	91	400	67	90
MV 500 × 2000 B	8.7	38.2	85	374	63	84
MV 500 × 2000 A	12.4	54.4	122	534	90	120
MV 550 × 1000 B	5.3	21.0	52	206	38	46
MV 550 × 1000 A	7.6	30.0	75	294	55	66
MV 550 × 1500 B	8.0	31.5	78	309	58	69
MV 550 × 1500 A	11.4	45.0	112	441	82	99
MV 600 × 1000 B	6.3	22.8	62	224	46	50
MV 600 × 1000 A	9.0	32.6	88	320	65	72
MV 600 × 1500 B	9.5	34.2	93	336	69	76
MV 600 × 1500 A	13.5	48.9	132	480	98	108
MV 750 × 1000 B	9.8	28.7	96	282	71	63
MV 750 × 1000 A	14.0	41.0	137	402	101	90
MV 750 × 1500 B	14.7	43.1	144	423	106	95
MV 750 × 1500 A	21.0	61.5	206	603	152	135

Table 14.5. Continued

Element size H × L Compound A or B	Rated performance for one element					
	E (tonne/m)	*R* (tonne)	*E* (kN m)	*R* (kN)	*E* (ft/kips)	*R* (kips)
MV 800 × 1000 B	11.2	30.5	110	299	81	67
MV 800 × 1000 A	16.0	43.6	157	428	116	96
MV 800 × 1500 B	16.8	45.8	165	449	122	101
MV 800 × 1500 A	24.0	65.4	235	642	174	144
MV 800 × 2000 B	22.4	61.0	220	599	162	134
MV 800 × 2000 A	32.0	87.2	314	856	232	192
MV 1000 × 900 B	15.8	34.3	155	337	113	76
MV 1000 × 900 A	22.5	49.0	221	481	162	108
MV 1000 × 1000 B	17.5	38.1	172	374	126	84
MV 1000 × 1000 A	25.0	54.4	245	534	180	120
MV 1000 × 1500 B	26.3	57.1	258	560	189	126
MV 1000 × 1500 A	37.5	81.6	368	800	270	180
MV 1000 × 2000 B	35.0	76.2	343	748	252	168
MV 1000 × 2000 A	50.0	108.8	490	1068	360	240
MV 1250 × 900 B	24.6	42.8	241	420	177	95
MV 1250 × 900 A	35.1	61.2	344	600	253	135
MV 1250 × 1000 B	27.3	47.6	268	467	197	105
MV 1250 × 1000 A	39.0	68.0	383	667	282	150
MV 1250 × 1500 B	41.0	71.4	402	701	296	158
MV 1250 × 1500 A	58.5	102.0	574	1001	423	225
MV 1250 × 2000 B	54.6	95.2	536	934	395	210
MV 1250 × 2000 A	78.0	136.0	766	1334	564	300
MV 1450 × 1000 B	36.8	55.3	361	543	266	122
MV 1450 × 1000 A	52.6	79.0	516	775	380	174
MV 1450 × 1500 B	55.2	83.0	542	813	399	183
MV 1450 × 1500 A	78.9	118.5	774	1162	570	261
MV 1450 × 2000 B	73.6	110.6	722	1085	532	244
MV 1450 × 2000 A	105.2	158.0	1032	1550	760	348
MV 1600 × 1000 B	44.8	61.0	440	599	323	135
MV 1600 × 1000 A	64.0	87.2	628	855	462	192
MV 1600 × 1500 B	67.2	91.6	659	898	485	202
MV 1600 × 1500 A	96.0	130.8	942	1283	693	288
MV 1600 × 2000 B	89.6	122.1	879	1197	647	269
MV 1600 × 2000 A	128.0	174.4	1256	1710	924	384

Table 14.6. The rated performance for one MI fender element

Element size H × L Compound A or B	Rated performance for one element					
	E (tonne/m)	*R* (tonne)	*E* (kN m)	*R* (kN)	*E* (ft/kips)	*R* (kips)
MI 2000 × 1000 B	54.8	54.8	538	538	397	121
MI 2000 × 1000 A	89.7	89.7	880	880	649	198
MI 2000 × 1050 B	57.6	57.6	565	565	417	127
MI 2000 × 1050 A	94.1	94.1	923	923	681	208
MI 2000 × 1100 B	60.3	60.3	592	592	437	133
MI 2000 × 1100 A	98.6	98.6	967	967	714	217
MI 2000 × 1150 B	63.1	63.1	619	619	457	139
MI 2000 × 1150 A	103.2	103.2	1012	1012	747	228
MI 2000 × 1200 B	65.8	65.8	645	645	476	145
MI 2000 × 1200 A	107.6	107.6	1056	1056	779	237
MI 2000 × 1250 B	68.6	68.6	673	673	497	151
MI 2000 × 1250 A	112.1	112.1	1100	1100	812	247
MI 2000 × 1300 B	71.2	71.2	699	699	515	157
MI 2000 × 1300 A	116.6	116.6	1144	1144	844	257
MI 2000 × 1350 B	74.0	74.0	726	726	536	163
MI 2000 × 1350 A	121.1	121.1	1188	1188	877	267
MI 2000 × 1400 B	76.7	76.7	752	752	555	169
MI 2000 × 1400 A	125.6	125.6	1232	1232	909	277

Arch fenders

Table 14.7 shows the rated performance for one arch fender.

Table 14.7. The rated performance for **one arch fender**

Rated properties compound A				Fender type MA	Dimension in mm					
Energy absorbed		Reaction force		H × L	W	A	B	C	Anchor	Number of anchors
Tonne/m	ft/kips	Tonnes	kips							
0.6	4.4	10.3	22.7	150 × 1000				855		4
0.8	5.8	15.4	34.0	150 × 1500				675		6
1.1	8.0	20.4	45.0	150 × 2000	300	98	240	620	M 20	8
1.4	10.1	25.4	56.0	150 × 2500				785		8
1.7	12.6	30.4	67.0	150 × 3000				715		10
1.2	8.5	15.4	34.0	200 × 1000				900		4
1.7	12.6	22.8	50.2	200 × 1500				700		6
2.3	16.7	30.1	66.4	200 × 2000	400	130	320	630	M 24	8
2.9	20.7	37.5	82.6	200 × 2500				800		8
3.4	24.8	44.8	98.7	200 × 3000				725		10
1.7	12.6	17.1	37.7	250 × 1000				865		4
2.6	18.9	25.2	55.6	250 × 1500				680		6
3.4	24.8	33.2	73.2	250 × 2000	500	162	410	620	M 24	8
4.2	30.2	41.3	91.1	250 × 2500				790		8
5.0	36.2	49.3	108.7	250 × 3000				715		10
2.8	20.6	23.2	51.2	300 × 1000				900		4
4.2	30.2	34.1	75.1	300 × 1500	600	195	480	700	M 30	6
5.5	39.8	44.9	98.9	300 × 2000				630		8
6.8	49.3	55.7	122.7	300 × 2500				800		8
5.2	37.6	31.7	69.8	400 × 1000				900		4
7.5	54.6	46.1	101.7	400 × 1500	730	260	610	700	M 30	6
9.9	71.7	60.5	133.4	400 × 2000				630		8
12.3	88.8	74.9	165.2	400 × 2500				800		8
8.3	60.0	40.5	89.3	500 × 1000				900		4
12.0	86.7	58.6	129.1	500 × 1500	894	324	750	700	M 36	6
15.7	113.5	76.5	168.7	500 × 2000				630		8
12.2	88.2	49.7	109.6	600 × 1000				900		4
17.5	126.6	71.3	157.3	600 × 1500	1020	390	876	700	M 36	6
22.8	164.9	92.9	204.9	600 × 2000				630		8

Further reading

British Standard BS 6349 (1994) *Part 4, Code of Practice for Design of Fendering and Mooring Systems*. London: BSI.

EAU (1996) *Empfehlungen des Arbeitsausschusses fur Ufereinfassungen (Recommendations of the Committee for Waterfront Structures, Harbours and Waterways*, 7th English version).

International Navigation Association, PIANC (2002) *Guidelines for the Design of Fenders Systems 2002*, Report of Working Group 33.
Ministerio de Obras Publicas y Transpotes (1990) ROM *Recomendaciones para Obras Maritimas (Maritime Works Recommendations, Actions in the Design of Maritime and Harbour Works* ROM 0.2-90. English version), Madrid.

15

Erosion protection

15.1 General

The erosion of the sea bottom in front of a berth structure and of the filling under an open berth structure will generally be due to the wave actions at the upper part of the filling and from propeller current from the main ship propellers and/or the bow and stern thrusters at the lower part of the filling and of the sea bottom, as illustrated in principle in Fig. 15.1.

The introduction of the ship's bow and stern thrusters around 1960 was due to the need to increase the ship's manoeuvrability and thereby minimize its manoeuvring time in the port. New and powerful ships with modern propeller systems, frequently combined with aggressive manoeuvring, can cause severe erosion in ports that otherwise would have remained stable for decades. The larger passenger ships can have two or three bow thrusters, the container ships usually have one, the ro/ro ships and ferries may have one bow and one stern thruster and two main propellers.

The most severe erosion effects on under-berth slopes or the sea bottom under a ship are the erosion from the main propeller and bow thrusters from container ships, ro/ro ships and ferries.

The intersection point between the toe of the slope and the sea bottom should be set back approximately 1 m behind the berth line as shown in Chapter 11 to ensure that no stone has been placed outside the theoretical slope line, or has fallen down to the bottom where it can damage the hull of a ship.

It is generally recommended that when one designs the seabed erosion protection it will be cheaper in the long run to accept some

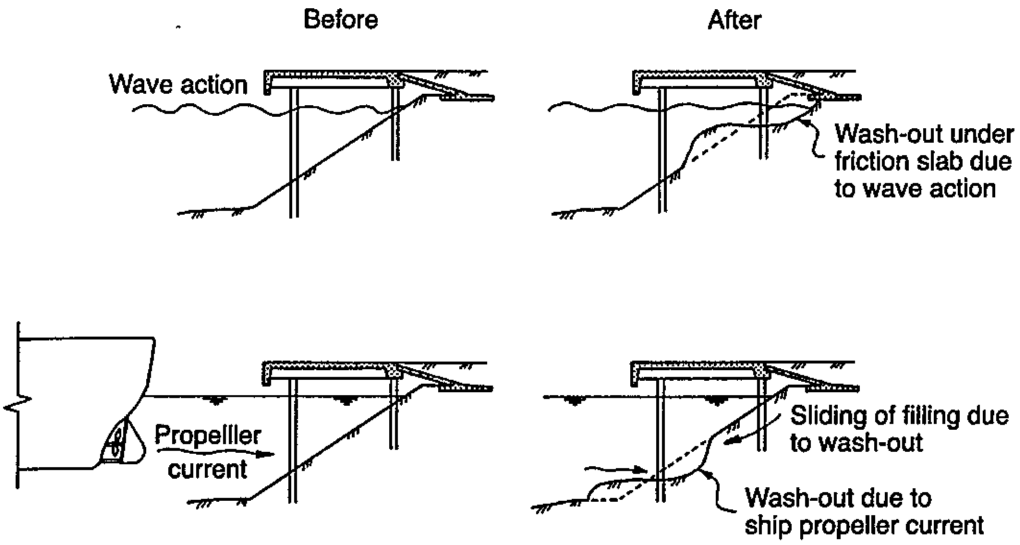


Fig. 15.1. Erosion due to wave action and ship propeller current

damages during the lifetime. But due to difficulty of access under an open berth after the completion of the deck structure, it is recommended that for under-berth slope protection this should be designed to be maintenance free.

The erosion of the sea bottom from the main propeller will depend upon whether the berthing or unberthing operations take place at low or high tide; but in the design of the protection one should always assume low tide.

Many factors must be taken into consideration when design work is undertaken. Examples of variables are type of seabed, depth and slope, type of berth construction, characteristics of vessels used (type of propeller, engine size), frequency of arrivals and departures, angle of approach, etc.

Seldom does the designer know the characteristics of the ships, that will use the berth structure during its service life. Table 15.1 shows some figures for the diameter and power of the main propeller and the bow thrusters for container ships for an approximate accuracy of ±10 per cent.

Special care should be taken, because of the ship's propeller action and its effect on the harbour bottom, and on the slope of the filling under the berth structure, particularly during berthing and unberthing by the ship. The action of the ship's propeller is the prime eroding factor with speeds of up to 4 and 8 m/s near the harbour bottom compared to, for example, the tidal currents, which are around 1–2 m/s. The propeller currents are due to:

Table 15.1. Diameter and power of propeller

Ship size in dwt	Main propeller		Bow thruster	
	Power (kW)	Propeller diameter in mm	Power (kW)	Propeller diameter in mm
10 000	8000	4500	500	1700
20 000	15 000	5500	750	1800
30 000	20 000	6500	1000	2000
40 000	26 000	7000	1200	2200
50 000	33 000	7400	1400	2300
60 000	40 000	7500	1700	2500

(a) The stern propeller or screw will cause an induced jet current speed directly behind the propeller.
(b) The bow thruster consists of a propeller, which works in a pipe and is located crosswise to the longitudinal axis of the ship. A typical bow thruster has a diameter between 1.5–2.5 m. It is used for manoeuvring out from the berth line. Current velocities up to 7.0 m/s can be expected for bow thrusters of large, for example, container ships with propeller output up to about 1700 kW and with propeller diameter of 2.5 m. See Table 15.1. The thrust of the bow thrusters is in the range of about 3–15 tonnes.

15.2 Erosion due to wave action

The design of the erosion protection due to wave action at the front of the stone filling is generally based on the Hudson formula:

$$W_{50} = \frac{\rho_s \times H_{des}^3}{K_D \times \left(\frac{\rho_s}{\rho_w} - 1\right)^3 \times \cot\alpha}$$

where

W_{50} = average block weight in kN
H_{des} = design wave height, H_s to $1.4H_s$
ρ_s = specific gravity of block unit of quarry stone, 26 kN/m^3
ρ_w = specific gravity of seawater, 10.26 kN/m^3
α = slope angle of the cover layer
K_D = shape and stability coefficient of which berth front is 3.2, berth end or end of the filling under the quay is 2.3. For quarry stone and breaking waves berth front is 2.7.

The block weight W_{max} should be less than (3.6–4.0) × W_{50} and W_{min} should be greater than (0.2–0.22) × W_{50}.

If the berth structure is exposed to extreme wave conditions, the slope protection should be checked for stability due to wave action down to at least a depth equal to 2 × H_s.

The equivalent rock or stone diameter will be:

$$d_{equ} = \sqrt[3]{\frac{6 \times W}{\pi \times \rho_s}}$$

where d_{equ} is the equivalent rock or stone diameter in m; W is the bock weight in kN and ρ_s is the specific gravity of block unit of quarry stone, 26 kN/m^3.

15.3 Erosion due to the main propeller action

The exact design of the erosion protection against the actions of main ship propellers and the bow and stern thrusters is as the evaluation and comparison between the EAU 1996 and the PIANC working group shows, very difficult. This will be due to the fact that the size and type of the protection will depend on the velocity of the propeller current, which again will depend on the ship engine power, the speed, and the shape and the diameter of the propeller. To get all this information for all the different ships calling at the berth will be impossible. Experience, however, has shown that if the erosion protection against the propeller action has the same stone sizes and filter as that for the wave action from a design wave height of about 1.5–2.0 m, this will in most cases give sufficient erosion protection.

The erosion action due to propeller action on the bottom seabed with stone filling is, in principle, shown in Fig. 15.2.

The jet velocity caused by the rotating main propeller, called induced jet velocity, which occurs directly behind the main propeller, is recommended both by EAU 1996 and PIANC Working Group 22 to be calculated by the simplified formula:

$$V_{OM} = 0.95 \times n \times D_p$$

where V_{OM} is the initial centreline jet velocity from main propeller, n is the propeller revolutions/s and D_p is the propeller diameter.

If the output of the main propeller is known instead of the velocity, the induced jet velocity can be calculated as follows:

$$V_{OM} = c \times \left[\frac{P}{\rho_O \times D_p^2}\right]^{1/3}$$

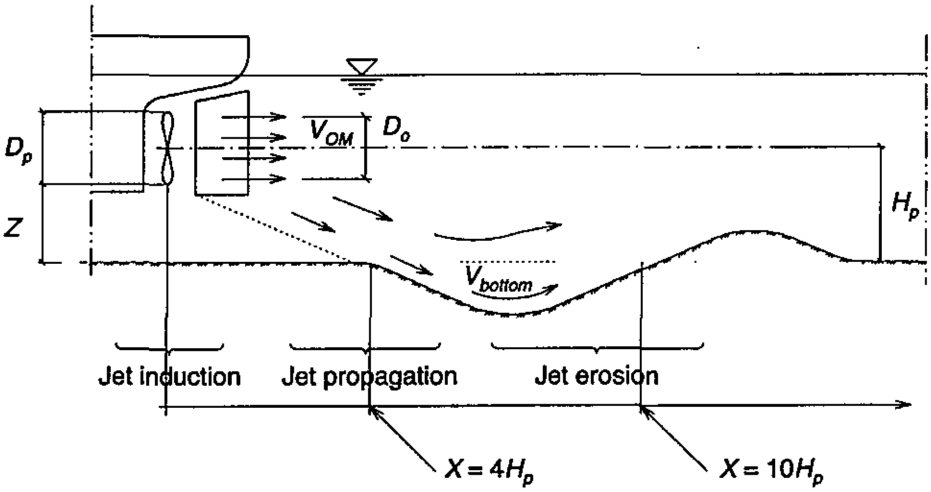


Fig. 15.2. Erosion of sea bottom due to the main propeller

where

V_{OM} = the initial centreline jet velocity from main propeller
c = 1.48 for free propeller or non-ducted propeller
1.17 for propeller in a nozzle or ducted propeller
P = the engine output power in kW
ρ_O = the density of seawater 1.03 t/m^3
D_p = the propeller diameter.

It is most unusual to use the main propeller at full power during berthing and unberthing except in the case of ferries. Generally practical experiences have shown that the machine power output for port manoeuvring lies between the following approximately values:

(a) 30 per cent of rated velocity for slow ahead
(b) 65–80 per cent of the rated velocity for half-speed ahead.

It is recommended that a speed corresponding to 75 per cent of the rated velocity be used for the design of the bottom erosion protection. For particularly critical conditions with high wind and/or current forces acting on the ship, the rated velocity or increased velocity at maximum power output must be assumed.

The propeller velocity expands cone-shaped from the propeller and loses velocity with increasing distance from the propeller. The zone of maximum seabed velocity V_{bottom}, which is essentially responsible for the erosion, is approximately a distance $4 \times H_p$ to $10 \times H_p$ from the propeller, as shown in Fig. 15.2. Therefore the most important parameters for the erosion of the seabed are the under keel clearance and the particle size of the seabed materials.

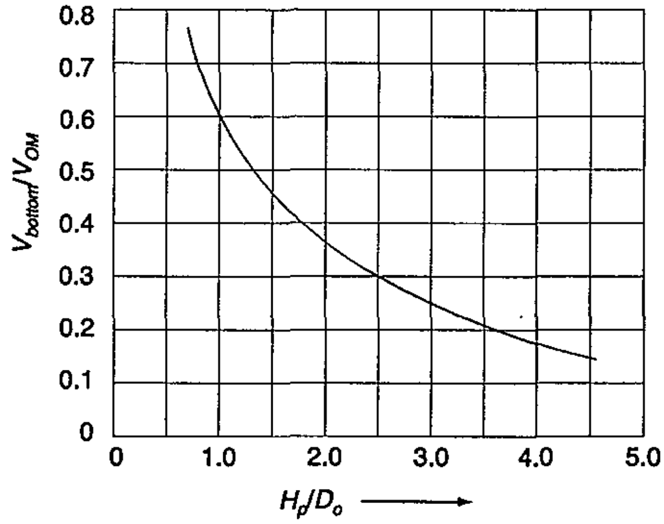


Fig. 15.3. Bottom velocity after PIANC

The EAU 1996 recommend that the seabed velocity can be calculated from the following formula:

$$V_{bottom} = V_{OM} \times E \times \left[\frac{H_P}{D_p}\right]^a$$

where

V_{bottom}	=	the bottom velocity due to the main propeller in m/s
V_{OM}	=	the initial centreline jet velocity from main propeller
E	=	0.71 for single-propeller ship with central rudder
		0.42 for twin propeller ship with middle rudder
H_P	=	the height of the propeller shaft over bottom
D_p	=	the propeller diameter
a	=	−1.00 for single-propeller ship
		−0.28 for twin-propeller ship.

The PIANC Working Group 22 recommend the following, Fig. 15.3, for the bottom velocity. For the calculation of the initial jet diameter D_o, PIANC recommend the following relationship:

(a) Non-ducted propeller $D_o = 0.71 \times D_p$
(b) Ducted propeller $D_o = D_p$

Due to later possible maintenance-dredging in front of the berth structure, the level or top of the bottom protection requires careful study because the erosion protection installed could be damaged during the maintenance dredging operations. It is therefore recommended that the protection layer should be placed at least 0.75 m below the lowest permitted dredging level.

15.4 Erosion due to the thrusters

The bow and stern thrusters are used for easier manoeuvring inside a narrow port area and/or during berthing and unberthing operations. The thrusters consist of a propeller installed in a tube and are located cross-wise to the longitudinal axis of the ship. They are always installed near the bow and sometimes also at the stern. When the thrusters are used in the berthing operation, they will generate a water current that will hit the quay front or the slope below an open berth structure directly and be diverted to all sides from there. If the water current hits a vertical berth front, e.g. a steel sheet pile wall, a part of the water current will hit the sea bottom and can cause erosion in the immediate vicinity of the berth wall.

The erosion action due to the bow thrusters' action of the seabed in front of the berth wall and/or the slope under the open berth structure is, in principle, shown in Fig. 15.4.

The jet velocity from the outlet of the bow thruster, can be calculated by the simplified formula:

$$V_{OB} = 1.04 \times \left[\frac{P}{\rho_O \times D_B^2}\right]^{1/3}$$

where

V_{OB} = the initial centreline jet velocity from the bow propeller in m/s
P = the bow engine output power in kW
ρ_o = the density of water 1.03 t/m
D_B = the inner diameter of the bow thruster opening in m.

For the design of the erosion protection due to bow thrusters one should assume that the ship use full power of the bow thruster when berthing and unberthing. For large container ships the jet velocity

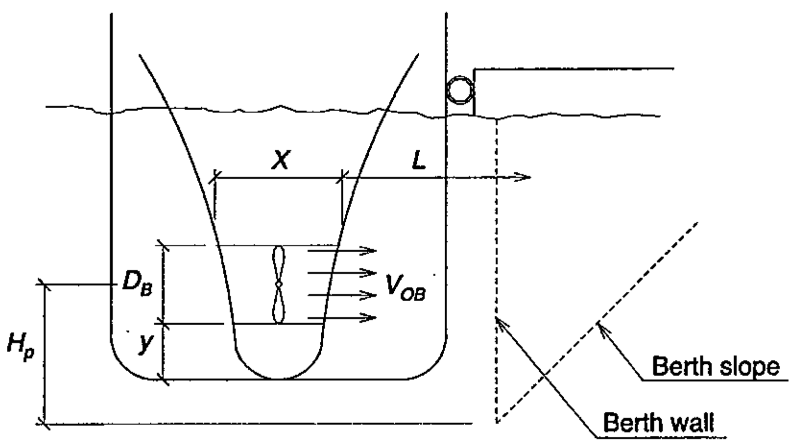


Fig. 15.4. Erosion against the berth structure due to the bow thrusters

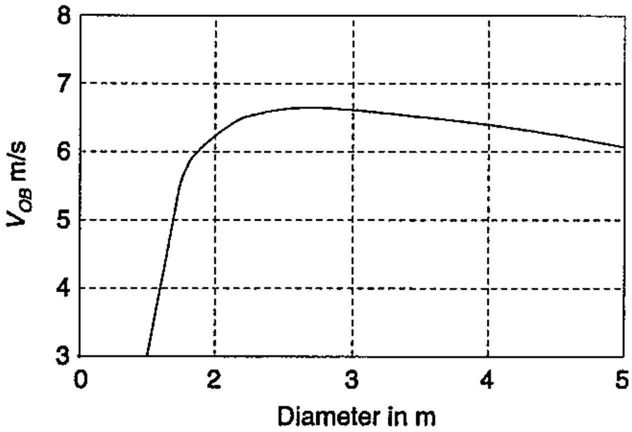


Fig. 15.5. Jet velocity of bow thrusters

from the bow thruster can, at full power, be assumed to be about 6.0–7.0 m/s. The PIANC Working Group 22 gives the initial jet velocity under full power for various thrusters diameters, as shown in Fig. 15.5.

The water jet velocity that will hit the bottom or the slope below the berth can be assumed to be:

$$V_{bottom} = V_{OB} \times 2.0 \left[\frac{D_B}{L}\right]$$

where

V_{bottom} = the bottom velocity due to the bow thrusters in m/s
V_{OB} = the initial centreline jet velocity from the bow propeller in m/s
D_B = the inner diameter of the bow thruster opening in m
L = the distance from the opening of the bow thruster to the berth wall in m.

For the design purpose the length x of the thruster tube, as shown in Fig. 15.4, can be taken as approximately 30 per cent of the beam of the ship, and the height y of the bow thruster above the keel can be taken as approximately equal to D_B. The distance L from the opening of the bow thruster of the ship should be to the berth wall or the slope.

15.5 The required stone protection layer

The EAU 1996 recommend the following formulas for required stone diameter for stability against propeller current:

$$d_{req} \geq \frac{V_{bottom}^2}{B^2 \times g \times \frac{(\rho_s - \rho_o)}{\rho_o}}$$

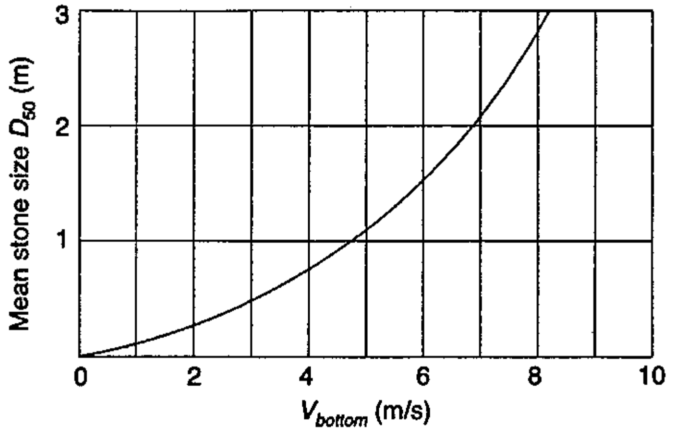


Fig. 15.6. Stone size for given bed velocity

where

d_{req} = the required diameter for the stones in m
V_{bottom} = the bottom velocity in m/s
B = the stability coefficient:
0.90 for ship without central rudder
1.25 for ship with central rudder
g = the acceleration of gravity 9.81 m/sec^2
ρ_s = the density of stone 2.65 t/m^3
ρ_o = the density of water 1.03 t/m^3.

The PIANC Working Group 22 recommend the following, Fig. 15.6, for the mean stone size D50 in m for no erosion for the given bed velocity.

The equivalent rock weight will be:

$$W = \frac{d_{equ}^3 \times \pi \times \rho_s}{6}$$

where

W = the bock weight in kN
d_{equ} = the equivalent rock or stone diameter in m
ρ_s = the specific gravity of block unit of quarry stone, 26 kN/m^3.

15.6 Erosion protection systems

For the protection of the sea bottom and the filling from erosion, the following protection systems are in use:

(a) Rock blocks or stones and rip-rap placed on a filter layer of gravels and/or a filter fabric.
(b) Filling loose stone with grouting.

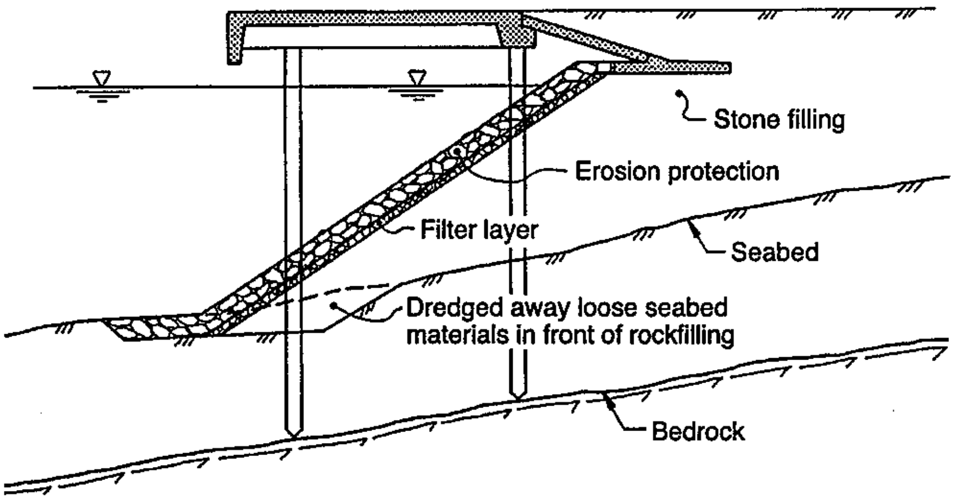


Fig. 15.7. Erosion protection of stone filling

(c) Covering with reinforced concrete slabs.
(d) Covering with flexible composite systems.

The most common erosion protection systems are (a) and (d).

The erosion protection with **rock blocks or stones** fill under an open pile berth structure is illustrated in Fig. 15.7. The erosion protection layer and thickness will depend upon the current velocity, the angle of the slope and the coarseness of the materials in the front of the filling. This protection system is one of the most frequently used, and the requirements shown in Fig. 15.7 must be met.

The natural inclination of the rock fill is approximately 1:1.2 when constructed. The final slope of the fill is usually recommended not to be steeper than 1:1.5 when carried out underwater, or the slope should not be steeper than the acceptable safe geotechnical stability of the slope. This can be obtained by reworking the slope with excavating equipment or by dumping from a barge.

The thickness of the erosion protection layer should be more than $3 \times d_{50}$ or $1.5 \times d_{max}$, and the layer should not be less than about 1.0 to 1.5 m. A thickness of two layers of rock is recommended. The smallest rock size to be used for the primary rock layer should be approximately 500 kg–1000 kg. In addition to quarry stone, the following materials can also be used as protection of, for example, reinforced concrete units such as Tetrapods and Dolos, etc. Between the stone filling and the protection layer a filter layer should be constructed.

The difference between rock or stone armour and rip-rap is that rock armour has a narrow range of sizes and must be placed individually, whereas the rip-rap contains a large or wide range of sizes and is placed by

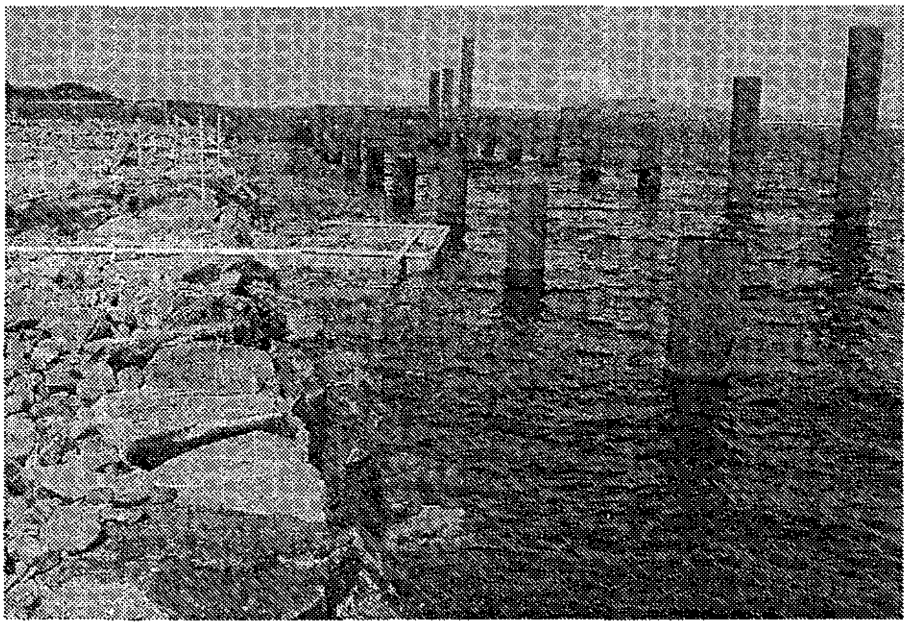


Fig. 15.8. Detail of the rock erosion protection before construction of the berth itself

dumping. The rock layer derives its stability due to interlock by its method of placing unit by unit, whereas the rip-rap get its stability from the packing effect due to the wide range of sizes. When the stones are over approximately 500 kg it can only be placed properly by a grab.

The thickness and the size of the stones in the filter layer will depend on the materials in the core stone filling. A detail of the rock or stone erosion protection is shown in Figs 15.8, 15.9 and 15.10. A general empirical rule is that the weight of the rock in the second layer should not be smaller than $\frac{1}{10}$ to $\frac{1}{15}$ of the weight of the rock in the primary armour layer. Between the filter layer and the stone filling, a geotextile matt could be used to ensure no migration of finer particles from the stone filling.

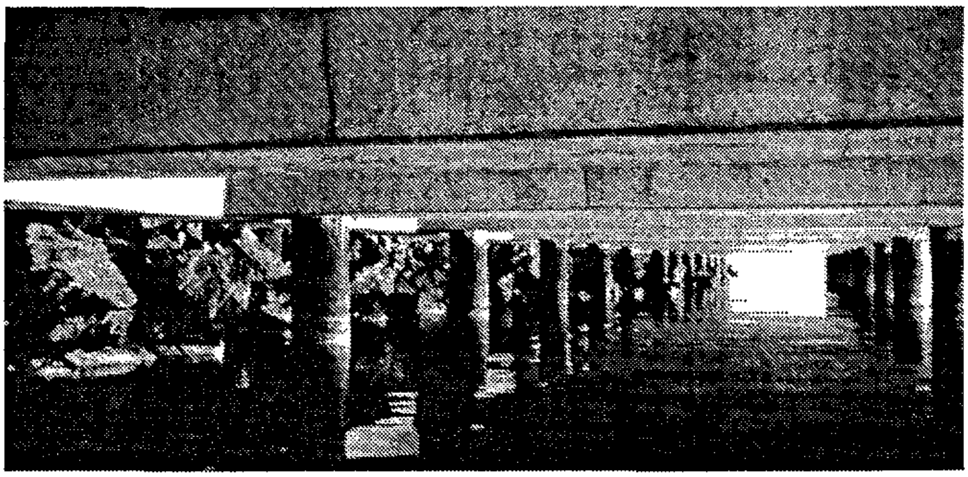


Fig. 15.9. Detail of the rock or stone erosion protection under the berth slab

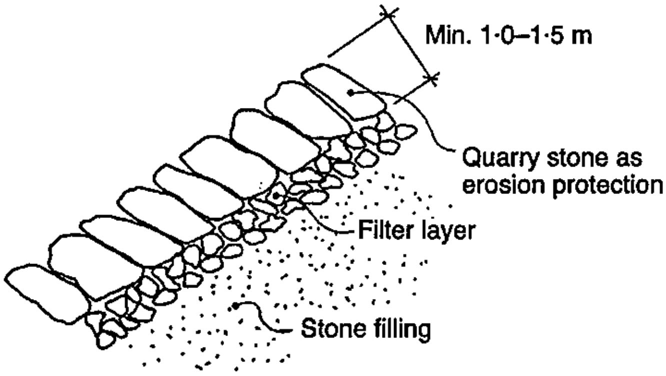


Fig. 15.10. Detail of rock or stone erosion protection

In the case of a soft seabed in front of the stone fill slope under an open pile berth structure, the erosion protection layer of the slope should be extended at least 3–5 m out in front of the berth line, as shown in Fig. 15.10. Since the slope under an open berth is more exposed to erosion than the horizontal sea bottom, it is recommended that the estimated D_{50} quarry stone or rip-rap is increased by 50 per cent.

Backfilling against or behind the berth structure itself should either be crushed or excavated rock with a maximum stone size of 300 mm. The filling above sea level should be compacted by performing at least 4 passes with a 600 kg vibrating plate over the area.

For bottom erosion protection of **loose stone cover**, the following requirements must be met:

(a) The installation of the stone should be in at least 2 layers.
(b) The stone cover should be stable to velocity due to the propeller action.
(c) There should be installed a filter layer of grain or a textile filter between the subsoil and the stone layers.

If quarried rock of the needed weight for rip-rap is not available, lighter rock **grouted** with concrete such that the porosity of the protection is retained could be used. It is recommended that the following aspects must be taken into consideration if the bottom protection of stone filling is grouted by, for example, concrete to make the erosion protection more stable against erosion:

(a) Depending on the grouting area, a minimum pore volume of about 15–20 per cent, which should be continuous from the bottom to the top surface of the fill to compensate for any hydrostatic pressure, if necessary.
(b) Grouted stone filling can be stable up to very high bottom velocity of approximately 7 m/s.

(c) As the grouting stone fill forms a stable but rigid unit, erosion can occur at the edges due to under washing because the grouting cannot react flexibly to these.

An ideal erosion protection could be an underwater in situ reinforced **concrete slab** because the thicknesses can be constructed with greater accuracy than a stone-filling layer. The underwater concrete slab can be constructed in thickness from approximately 30 cm and up to 80 cm or more depending on the concreting technique. The advantage of this system is that compared to a stone filling, the stones cannot be dislodged by the propeller action or by an anchor. The disadvantages of the system are the installation of concrete underwater, which could be a complicated and very costly process.

Erosion protection with flexibility composite systems, for example mattresses filled with concrete, such as the FlexiTex system from Norway or equivalent, should be placed on the prepared slopes and sea bottom when empty, joined together and then pumped full of concrete. The mattresses, which are made from double-weave mattress, have been woven together at regular points that act as filters to even out water pressure. When using double-weave mattresses with concrete infill it is important to level the supporting bed. The mattress can tolerate unevenness of up to $\pm 0.15\ m/m^2$.

The mattresses, which are supplied in widths of about 3.75 m and with lengths up to about 100 m, can be attached to panels in advance for rapid installation in the area to be protected. When calculating the required mattress size, one must remember that when the mattresses are filled with concrete they will shrink by approximately 10–15 per cent in both directions. The thickness of the mattresses can be produced from 7.5 cm up to 60 cm, with area weights when filled with concrete of approximately between 150–1200 kg/m^2. The mattresses for berth protection are usually delivered with a thickness of at least 20 cm to give an approximate weight/m^2 of ca. 500 kg.

The filling of the mattresses is shown in Fig. 15.11. The quality of the concrete used must be designed for pumping into mattresses. When the concrete is pumped into the mattresses this will cause the water to evacuate through the fabric. The fabric acts like a sieve and is designed to prevent concrete particles from getting through it.

The design of the concrete should be suitable for pumping through a pipeline of 50–75 mm tube dimension. From experience the proportion of concrete mixture/m^3 concrete should be approximately:

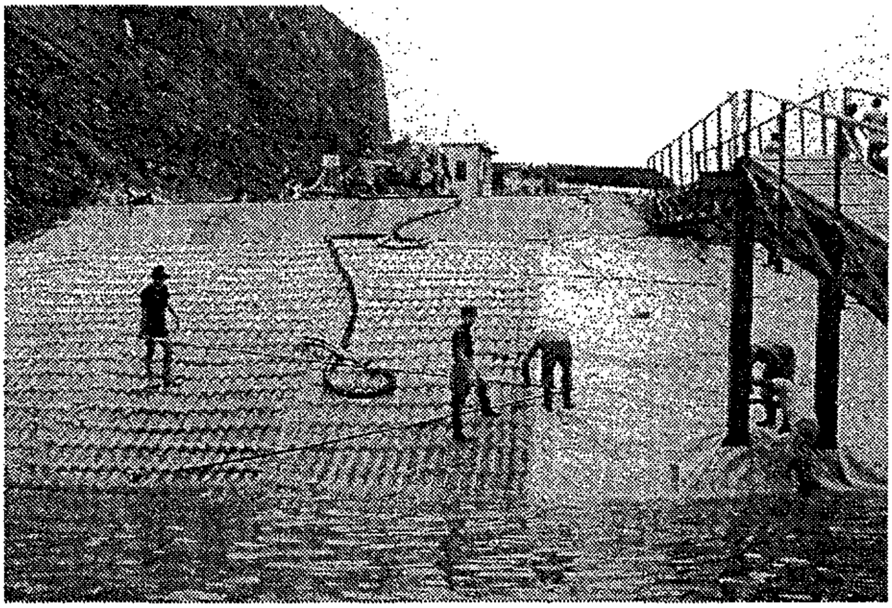


Fig. 15.11. Filling of the mattresses

Cement = 350–500 kg of standard Portland

Silica = 30–50 kg

Plasticizer = 5–15 kg

Sand = 1400–1600 kg with grain size between 0–8 mm. It is recommended that the sand is very fine to ease the pumping.

W/C ratio = between 0.4–0.5

The mattresses are usually fitted with industrial-quality zip fasteners for easy joining below water level. The result is a mass of pillow-like concrete units as seen in Figs 15.12 and 15.13. The method is fast to construct; the cost could be lower than for a rock protection system and they are considered durable.

When the sea bottom needs to be repaired or protected against erosion, a concrete mattress covering the area can be an excellent choice. It is essential that the repair work will be extended well outside the point where the erosion can start. As the weakest point where the erosion can start is at the edge of the mattress, it is therefore very important that the ending of the outer edge of the mattress assembly is secured down in a trench covered with concrete-filled bags or rocks or gravel, as shown in Fig. 15.14.

Where the propellers may act against a solid berth wall resulting in a strong downward current, the erosion protection layer should be extended a distance out from the berth wall as shown, in Fig. 15.14.

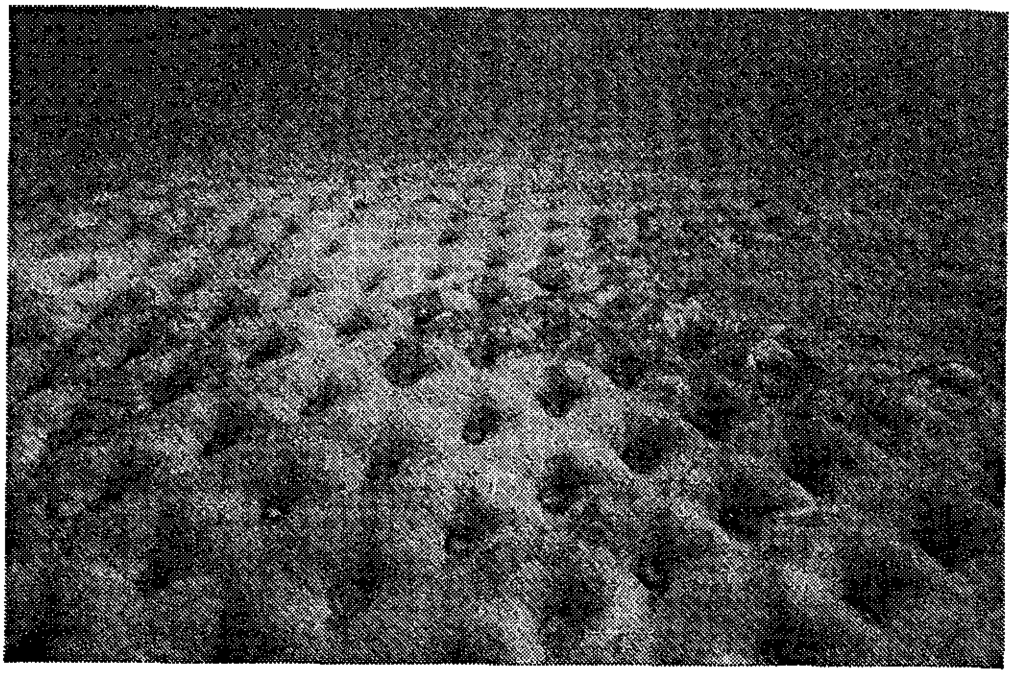


Fig. 15.12. Mattress filled with concrete

Depending on the seabed, the type of ship, etc., the distance should be approximately the width of the largest ship using the berth or at least 5 m beyond the longitudinal axis of the design ship. The depth X should be approximately 1 m more than the expected erosion.

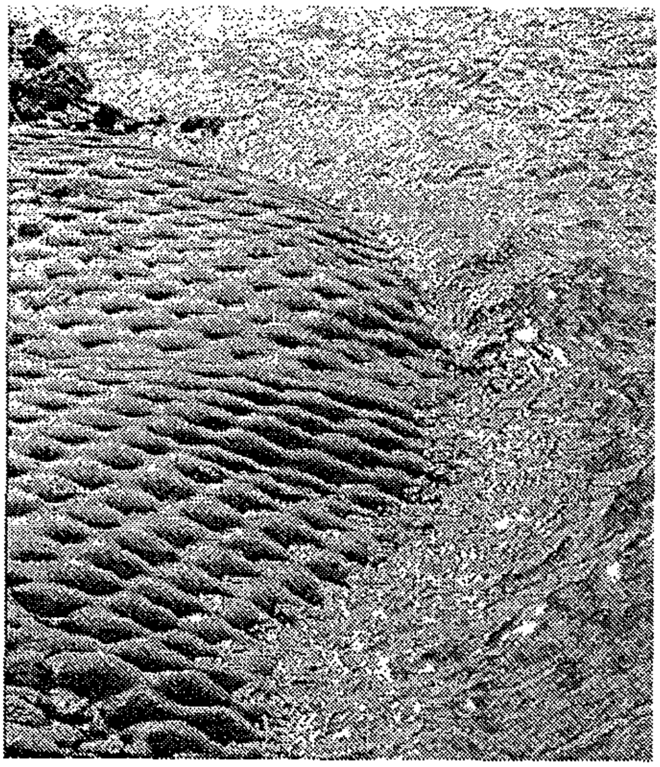


Fig. 15.13. Mattress filled with concrete

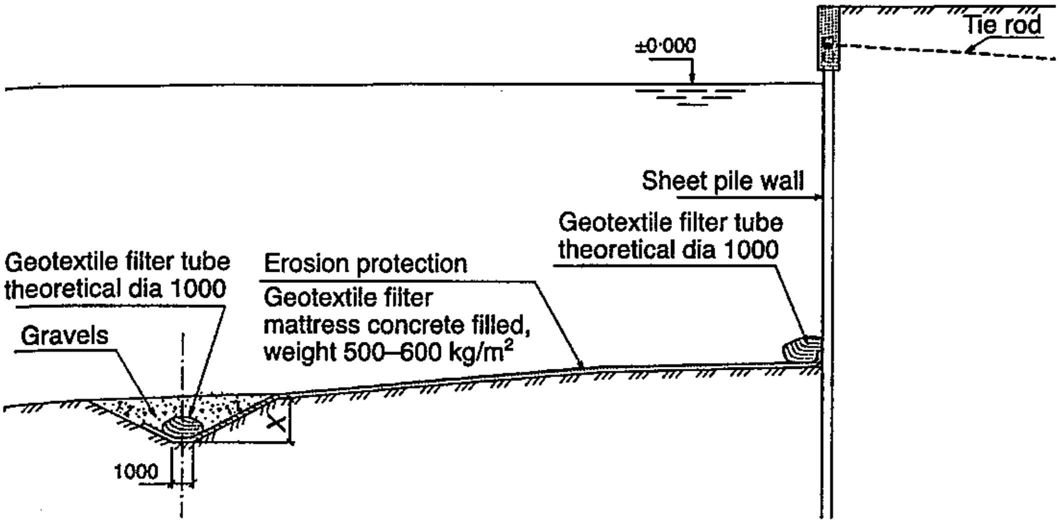


Fig. 15.14. Typical section of concrete mattresses

A different flexibility system is the wire box-like **Gabion system** filled with small rock stones. This system could be acceptable if the current speed is very slow. The disadvantage of the wire Gabion mats is that the wire boxes or mesh is liable to corrosion or that the wire could break due to movements between the different mats.

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16

Steel corrosion

16.1 General

The corrosion of steel sheet piles varies in differing conditions of sea air and seawater exposure. Experience has shown that severe corrosion occurs in saline water and under marine growth especially in the splash zone and the lower tidal zone with alternative wetting and drying. Another type of corrosion by sulphate-reducing bacteria has been found in the sea-bottom zone. These bacteria are active in waters containing nearly no oxygen, like the conditions found in some very polluted harbour basins. See Fig. 16.1, which shows the general pattern of steel corrosion in marine environment.

The steel would be subjected to a natural corrosion process when it came in contact with water and at the same time in the presence of oxygen. The material abrasion from corrosion will depend on the local hydrological conditions and on the local vertical position regarding the water line, which means that there will be different zones where corrosion forms. The degree of corrosion or rusting rate and intensity is decreasing with increasing layer of rusting thickness, unless the cover rusting layer is constantly destroyed by, for example, waves washing actions against the steel face.

Under aquatic conditions, the corrosion rate is directly proportional to the electric conductivity of the water. The conductivity of seawater is high, involving a higher corrosion rate than in fresh water. The corrosion protection of steel in seawater has to be evaluated separately in different zones. The corrosion rate is at its highest in the splash zone and immediately under the water level.

For reasons related to corrosion, the circular steel sheet pile cell constructions can be a better solution than traditional sheet pile

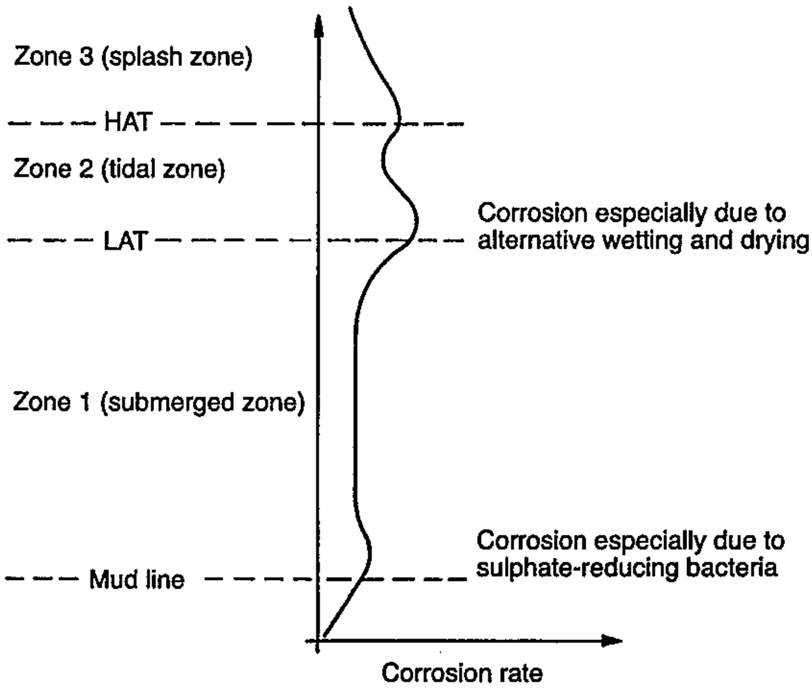

Fig. 16.1. Corrosion of steel sheet pile

walls. The circular steel sheet pile cells are heavily strained, a little over the sea bottom, where the maximum tensile forces from the fill are acting. Whereas in a sheet pile wall the profiles have to resist great moments due to the loads in the tidal zone where the anchors are connected to the wall.

Whether the corrosion is acting on both sides or only on the outside of the steel profiles depends very much on the kind of fill used in the steel pile cells or behind the sheet pile wall. If dense material like sand and gravel is used, corrosion on the outside can only be assumed, whereas rock fill leaving water pockets behind the sheet piles implies danger of corrosion on both sides. One should therefore specify and check that a fill of at least 1 m thickness close up to the sheet piles is sand or gravel.

The rate of corrosion will depend on the following:

(a) Atmospheric conditions of the environment.
(b) Seawater salinity. Normally the dissolved salts concentration of seawater lies between 3.2 and 3.6 per cent. The maximum corrosion rate occurs when the salt concentration is a little lower, at around 2.5–3.0 per cent. This is typically found in estuarial locations where the river freshwater mixes with the seawater.
(c) pH value of the seawater. If the pH value is less than 4, the rate of corrosion will increase dramatically.

(d) Dissolved oxygen. If the dissolved oxygen content in the seawater increases, the rate of corrosion will also increase.
(e) Temperature. The rate of corrosion increases in direct relation to the increase of the temperature.
(f) Wave and current. The rate of corrosion increases in direct relation to the wave action against the steel structure and the current speed.
(g) Chemical composition of the stratum into which the steel is to be embedded.

The corrosion protection of steel piles will vary according to their ambient conditions. The corrosion is generally taken into account as a corrosion allowance for the material thickness. The extent of the corrosion allowance depends on the planned design working life of the structure and on the estimated corrosion rate. The corrosion in soil is usually so low and uniform in the different soil layers that the protection of the steel is achieved simply by slightly over-dimensioning steel thickness.

16.2 Corrosion rate

In the absence of accurate corrosion recordings it can be assumed that, as a rule of thumb, the average corrosion of steel structural elements in berths amounts to about 0.10–0.15 mm/year per waterside of the steel sheet pile. The pitting corrosion rate in the tidal zone can in Scandinavian harbours be up to 0.5 mm/year, with an average of about 0.3 mm. In tropical waters the rate of corrosion is usually higher. The *Eurocode* recommends the following corrosion allowances under normal conditions, as shown in Table 16.1.

16.3 Corrosion protection systems

The reason that metal corrodes in the seawater tidal zone is due to the fact that parts of the metal surface act as anodes and other parts act as cathodes. Where the electrical current leaves the metal surface, the corrosion attack will start. Pitting corrosion can be dangerous if a pit has been formed, because the chemical composition of the electrolyte in the pit can accelerate the corrosion in the pit.

Where there is great uncertainty about the rate of corrosion in the environment of the berth structure, preparations should be made during construction for the later installation of cathodic protection, which is an electrochemical method of corrosion control. By installation of cathodic protection the corrosion of steel completely immersed

Table 16.1. Recommended corrosion allowances in mm under normal conditions

Soil conditions	Design working life (years)				
	5	25	50	75	100
Undisturbed natural soils (sand, silt, clay, schist, etc.)	0.00	0.30	0.60	0.90	1.20
Polluted natural soils and industrial grounds	0.15	0.75	1.50	2.25	3.00
Aggressive natural soils (swamp, marsh, peat, etc.)	0.20	1.00	1.75	2.50	3.25
Non-compacted and non-aggressive fills (clay, schist, sand, silt, etc.)	0.18	0.70	1.20	1.70	2.20
Non-compacted and aggressive fills (ashes, slag, etc.)	0.50	2.00	3.25	4.50	5.70

Notes
The corrosion rate in compacted landfill is slower than in uncompacted landfill. The design for compacted landfill can be made using the corrosion allowances for non-compacted landfill divided by two. The values given are for guidance only.
Local conditions should be taken into consideration. The values given for 5 and 25 years are based on measurements. The other values have been obtained by linear extrapolation and are therefore on the safe side.
Source: European Committee on Standardization Draft prEN 1993-5. Eurocode 3: Design of Steel Structures. Part 5: Piling. CEN 1995

underwater (Zone 1) can be substantially eliminated, and corrosion of steel alternatively exposed to wet and dry conditions, the tidal zone (Zone 2) and the splash zone (Zone 3), can be significantly protected with an impressed current system in the tidal and splash zone. This installation must be carried out by companies specializing in corrosion protection, e.g. Corroteam A/S, Norway or equivalent companies.

Marine steel structures can be protected by the following two main types of cathodic protection systems, as shown in Fig. 16.2:

(a) The **sacrificial anode** system, which consists of a sacrificial anode immersed in the seawater (the electrolyte) and electrically connected to the marine steel structure (e.g. the berth structure). The protected surface of the marine steel structure will now act as a cathode.

The sacrificial anode system requires no external source of electrical power and is relatively easy to install and maintain, and it is an attractive system if the required protective current is not large. The system will, when properly installed, require very little attention and maintenance during its design service life. The anodes, that are used nowadays, have a design life of about 15–20 years.

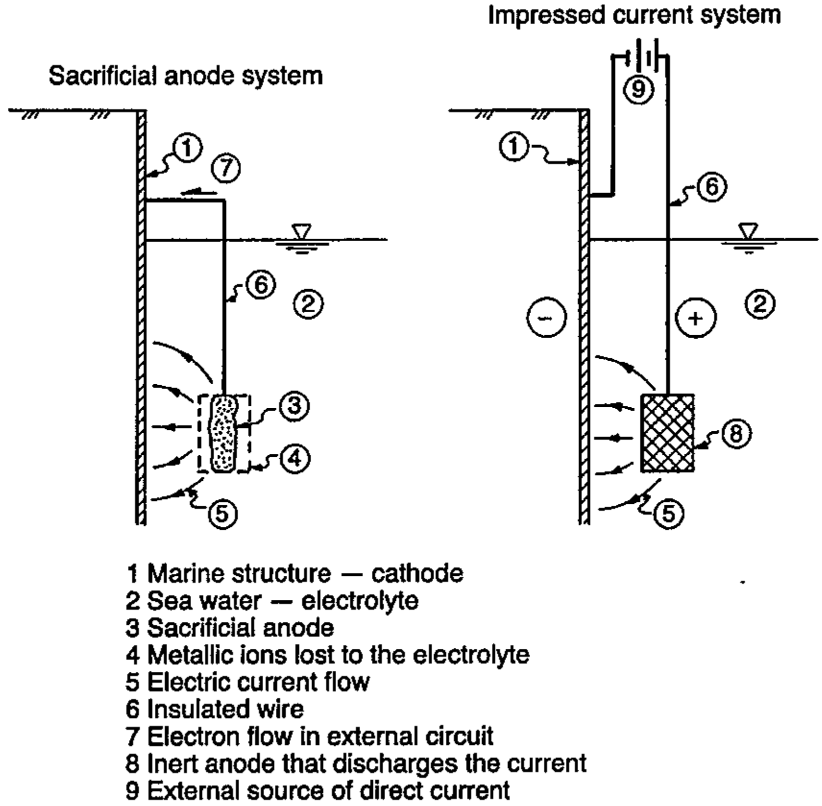

Fig. 16.2. Cathodic protection systems

When two different metals are coupled together in an electrolyte (e.g. seawater), one of the metals will act as a sacrificial anode and corrode and the other will act as a cathode. The one which will act as a cathode will be the most noble metal of the two. For example if steel and zinc are connected in seawater, the zinc will act as an anode to steel because a current will flow from the zinc to the steel. This reaction can only take place when there is an electrolyte present between the two dissimilar metals. The list below shows some metals' relationships to each other:

Protected end — cathodic or noble

gold platinum
nickel (passive)
copper
nickel (active)
steel, iron, cast iron
aluminium
zinc
magnesium

Corroded end — anodic

The anodes, that are used nowadays, are mostly anodes of aluminium alloy. The aluminium anodes give longer lifetime with less anode weight compared to zinc anodes.

(b) The **impressed current system** is used to protect marine steel structures, which require large quantities of current. As the name indicates, a protective direct current is impressed to the cathode surface by external means. A rectifier consisting of a step-down transformer and a rectifier stack converts alternating current to direct current.

The anodes used are inert anodes of platina, platinized titanium, lead alloys, magnetite or other suitable materials.

The difference between the impressed current system compared with the sacrificial anode system, is that in the sacrificial system the anodes corrode because the current is leaving their surface, while in the impressed current system the anodes could be made of non-corroding anode materials, which enable the anode to last much longer. For both the sacrificial and the impressed current system, the following must be considered to give the marine structure the desired service life:

(a) determine the required protective current
(b) determine the most suitable number, location, size and type of anode
(c) develop specifications for suitable mounting of the equipment to the structure
(d) develop specifications for proper maintenance inspections.

Instead of using a cathodic protection system, it is possible to **paint the steel** elements with anti-corrosion compositions or protective coatings to form a barrier to the environmental exposure and thereby delay the corrosion. The usefulness of this is often questionable because these barriers invariably break down after a number of years. Important factors in ensuring optimum performance of the protective coatings are the choice of coatings, the method of application and the thickness of coats. In Arctic harbours a coating system is generally not recommended because floating ice can destroy the coating.

The ideal and optimum protective system for steel in marine environment could theoretically be a combination of different protective systems, because one system, that is economical and effective in one zone, might not be suitable for another zone. For example some coatings are effective and economical in the splash zone, but less attractive in the submerged part of the structures due to difficult and high maintenance cost. In the submerged part the cathodic protection systems would be the most suitable. Therefore, if combinations of selected coatings and impressed current system are compatible, they can be an economical solution to the corrosion problem.

Where protective coatings or cathodic protection are not practical or their maintenance is doubtful, increased section or **extra thickness of steel** equal to the amount of corrosion expected for the lifetime of the berth structure may be economically justified and a technically better solution.

As a rough guide the steel thickness for steel used in marine structures should be a minimum of 10 mm where cathodic protection is not used and 6 mm where cathodic protection is used.

Further reading

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17

Underwater concreting

17.1 General

Placing of concrete in water is a very difficult operation. All aspects from mixing, transportation, placing and control of the work have to be carefully evaluated and should only be performed by very experienced engineers and workers. The aim in placing concrete underwater is to keep the fresh concrete and water apart as much as possible during the placing of the concrete, and to avoid a rapid flow of either of them when they come in contact, so that the cement will not be washed out. For these reasons, the correct placing method is the most important factor with respect to final quality.

Underwater concreting is not a new technique: it has been experimented with since about 1850. In 1910, the Norwegian, August Gundersen, took out a Norwegian patent on a 'Method of Underwater Casting for Concrete Columns and the like'. In the same year, the method was tried for the first time in Norway for underwater concreting of a reinforced structure. This method is, nowadays, the main underwater concreting method and is known as the tremie pipe method.

Since the 1980s, admixtures that increase the cohesion of the concrete and make direct contact with water possible without significantly changing the properties of the concrete have been developed and are widely used. The anti-washout (AWO) admixtures, e.g. Rescon T from Norway and similar products, have certain properties that influence the fresh concrete, and the setting and hardening of it. Knowledge about these properties is crucial for all parties involved.

17.2 Different methods of underwater concreting

17.2.1 General

A short summary of the most common methods for underwater concreting is given in the following sections.

17.2.2 Bucket concreting

The simplest way of placing underwater concrete in a formwork underwater is to lower the concrete through the water in an open bucket to a diver who will carefully place the concrete in the formwork. Bucket concreting should only be used for very minor and temporary work.

17.2.3 Sack concreting

This method is used in minor permanent works and repair works. The concrete is placed in porous sacks of woven materials and lowered down through the water to a diver. Since the sacks are only between 50–70 per cent full of concrete, the diver can push the sacks into shape to give them a good contact area with each other, either side by side and/or upon each other. Since the cement paste will be squeezed out through the woven sacks, a certain cementation will occur between the sacks. The opening of one sack should always be turned in towards another sack. To provide a stronger and a better result, the diver can drive reinforced steel bars through the sacks. The sacks are usually laid in bond similar to block walling.

17.2.4 Container concreting

The concrete is lowered down through the water in a closed bag or skip in one of the following ways:

(a) The bag method: where small amounts of concrete are required, for example in repair work, a canvas bag about 2 m long and about 0.5 m in diameter is a useful method for placing concrete underwater. The canvas bag, which is reusable, is filled with concrete and lowered to the specified location after the bag has been closed at both ends. Just above the casting spot the bottom of the bag is slowly opened, letting the concrete flow out of the bag into the form.

(b) The steel container or skip method: in this method a cylindrical steel container or skip is used with a top and bottom lid. This method is more effective than the bag method since it is possible to bury the bottom or the mouth of the skip in previously laid concrete and in this way prevent or reduce the possibility of wash-out of cement. When loaded, the skip should be full and a flexible cover or lid should be placed over the top opening. This will reduce the washout of cement during lowering and during discharging. The flexible cover will follow the top of the concrete down during pouring. To allow free flow of concrete through the skip, the skip should always be vertical during discharge of the concrete. The weight of the skip with the concrete will be sufficient to ensure that it sinks into the concrete surface. To reduce the possibility of washout the skip should be provided with a skirt. During pouring the skip should be slowly raised.

For concreting of small foundations underwater, a concrete with a cohesion-increasing admixture or AWO admixture will diminish the risk of washout of cement. In this case the skip concreting method could be a better alternative than the tremie pipe method.

17.2.5 Tremie pipe concreting

The concrete is transported and poured through the water by means of a rigid pipe that dips into the fresh concrete already placed. When the concreting starts, the first batch is passed through the pipe under the control of a sliding valve. The method is described in detail later in this chapter since it contains the fundamental principle of nearly all underwater concreting.

17.2.6 Hydrovalve concreting

This method is a refinement of the tremie pipe method, or it can be said to be a cross between the skip method and the tremie pipe method. Instead of using a rigid pipe, the concrete slides down a collapsible tube, which is kept closed by water pressure until the weight of the concrete in the tube overcomes the hydrostatic pressure and the tube skin friction. The concrete plug will then slide slowly through the tube, and the tube will be sealed behind each plug by the water pressure. A valve at the bottom end of the tube controls the concrete discharge.

17.2.7 Pump concreting

The pump concreting method can also be said to be an extension or a variation of the tremie pipe method. Instead of delivering the concrete into the formwork by the pressure created by the concrete's own weight, the concrete is placed into the formwork by hydraulic pumps, which pump the concrete through the pipe. Pump concreting is nowadays generally superior to other methods, especially in the case of concreting large concrete volumes or concreting in shallow water. If the pipeline is equipped with an outlet valve, the concrete pump method is both versatile and safe for many applications.

17.2.8 Injection

In this method the formwork is first filled with specially washed coarse graded aggregate. The voids in the aggregate are then filled by injection with a mortar or grout consisting of cement, sand and expanding and stabilizing material. This method can be especially useful in flowing water and in areas inaccessible to skips, tremie, hydrovalve or pump concreting such as undercuts, for example, under a foundation.

17.3 Tremie pipe method

17.3.1 General

The general principle of pouring concrete underwater by the tremie pipe method is shown in Fig. 17.1. The concrete is poured down a ridged pipe, usually of steel or plastic, from a hopper above the surface and pressed into the mass of concrete in the formwork by the weight of concrete in the pipe. If plastic pipes are used the strength must be assured to be adequate for the actual water depth. The pipe and hopper are suspended from a staging and mounted so that the steel pipe and hopper can be smoothly lifted and lowered vertically and independently of waves and tidal variations.

The height of the hopper above the water will depend on the needed casting pressure and the length of the tremie pipe. The pipe diameter should be between 15 and 30 cm. Nowadays 20 cm is the commonest. Reusable steel pipes should be built up of lengths from 1–2 m with watertight joints, such as bolted flanges with rubber gaskets. The pipes must be watertight and well cleaned. The lowest section of the pipe should have no flange at its lower end. Each pipe length should be easy to unscrew and remove. Figure 17.2 shows the tremie pipe

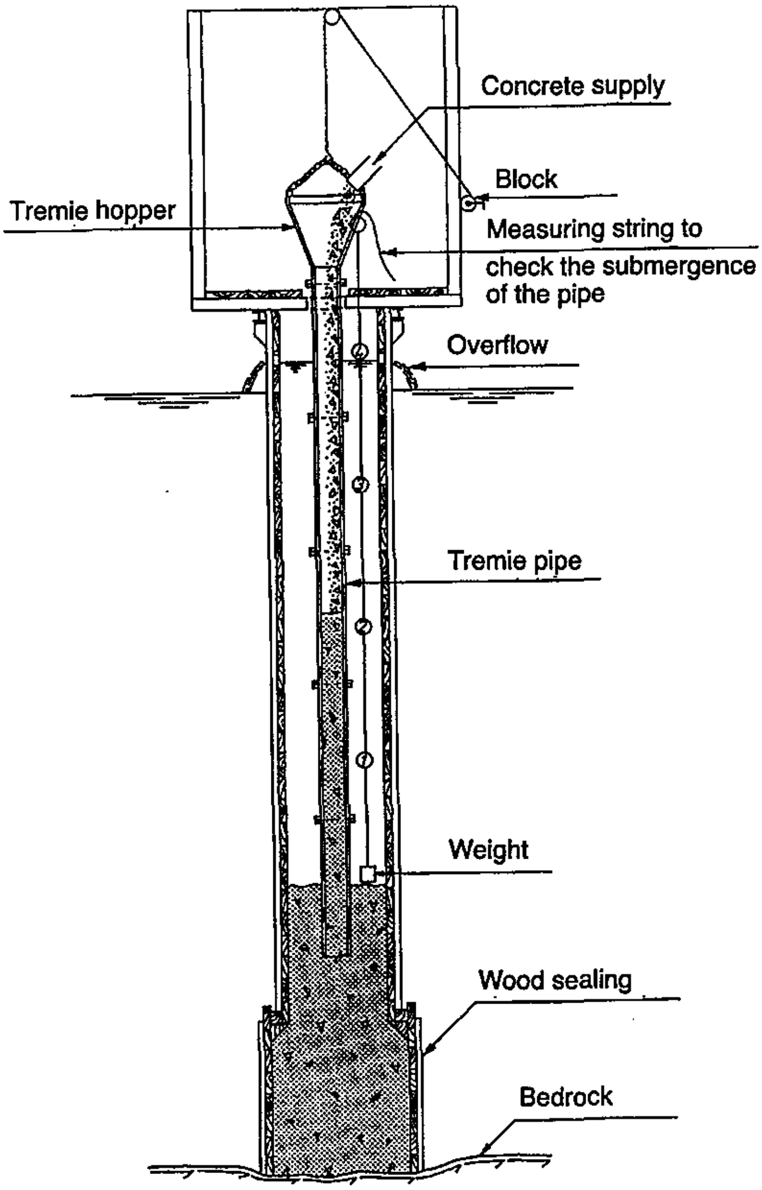

Fig. 17.1. Tremie pipe method

being lifted into and down the column formwork. Figure 17.3 shows the tremie pipe in the centre of the column formwork together with the column reinforcement, and Fig. 17.4 shows the complete arrangement just before the start of the concreting work.

17.3.2 Formwork

The formwork should be watertight to prevent water flow through the formwork with the possibility of washing the cement out of the fresh concrete. When wooden formwork is used, the boards should be tongued and grooved. Ordinary shuttering boards should only be used

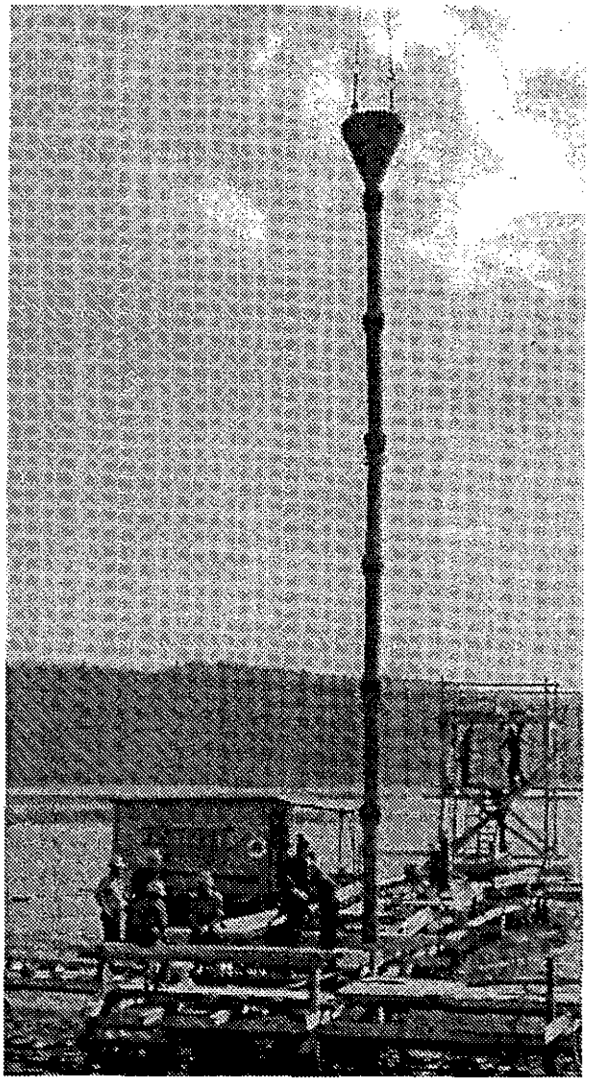

Fig. 17.2. The tremie pipe being lifted into the formwork

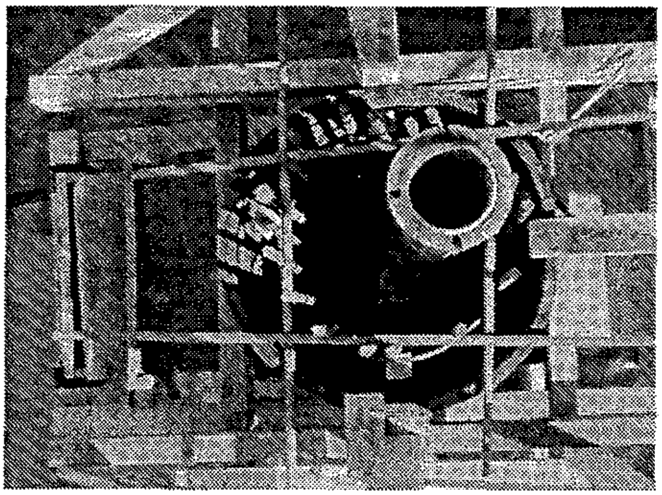

Fig. 17.3. The tremie pipe in the centre of the formwork together with the reinforcement

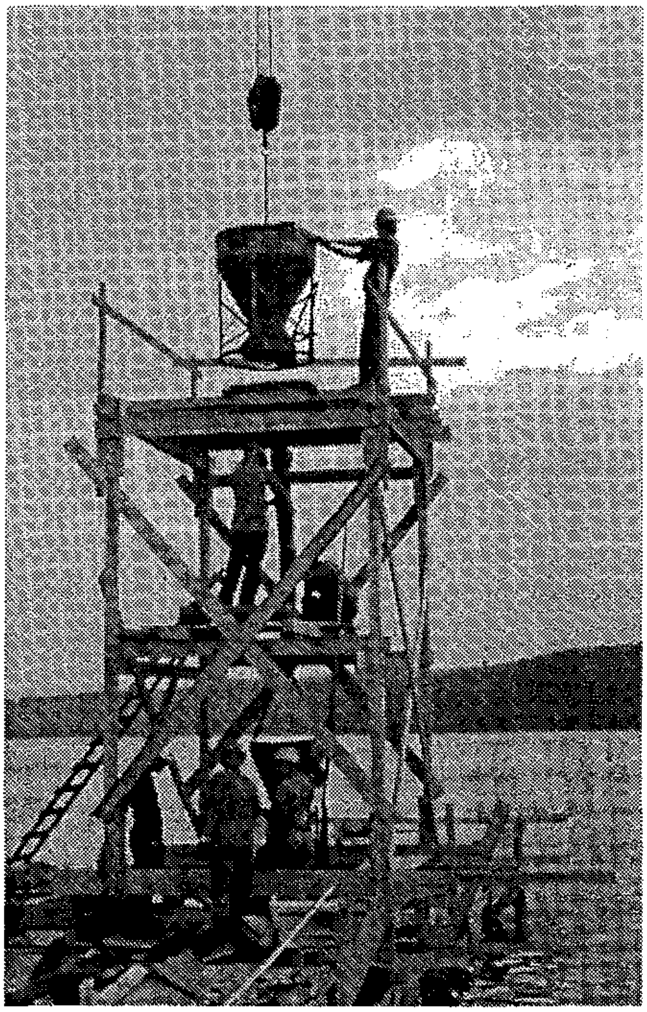

Fig. 17.4. The complete arrangement

in massive concrete structures in water without current. An overflow should be provided just above the waterline for the water displaced by the concrete.

The formwork should be either adjusted to the shape of the rock footing or sealed by other means, as shown in Fig. 17.1. Before the concrete work starts, a diver should check and seal possible leakages between the formwork and the rock. When wooden formwork is used, the formwork must be weighted or anchored, and attention should be paid to vertical uplifting forces against formwork surfaces, which are not vertical. Column bases on rock should be enlarged by at least 10 cm in all directions. The enlargement of the column base will result in an increase in buoyancy of the formwork, which must be taken into account.

The formwork must generally be robust, simple and easy to assemble or dismantle by divers or frogmen. Tie rods must be placed where they will not obstruct the movement of the tremie pipes or the flow of the concrete in the formwork.

All formwork, except the formwork in the tidal zone, should be removed in order to facilitate detailed inspection and control of the concrete. Especially, the foundations of columns, walls, casting joints and expansion joints should be carefully examined for any defects.

17.3.3 Spacing of tremie pipes

The horizontal distance to be poured from one pipe with a diameter of about 20 cm should not exceed approximately 2.5 m. The supply of concrete must be regular in order to assure a satisfactory form-filling rate. If these requirements cannot be satisfied, the area should be sectioned by means of partition walls or, if the capacity of the batching plant is large enough, two or more pipes can be used simultaneously. Alternatively anti-washout (AWO) concrete could be used.

The number of pipes or sectioning will also depend on the vertical tolerance required of the finished top surface. Generally the spacing between the pipes will be about 4–5 m. The concrete will flow about 2.5 m horizontally when using rounded gravel, and about 2.0 m horizontally when using suitable crushed stone. The slope of the concrete surface is likely to be in the range from 1:6–1:9 unless the concreting rate is very high or AWO concrete is used. A closer spacing will give a more level top surface.

17.3.4 Pouring of concrete

At the start of the concreting operation, the pipe will be full of water. The pipe should be lowered to the bottom of the formwork. The plug is then placed just above the water level and the pipe and the hopper are filled, as shown in Fig. 17.5.

The controlled lowering of the concrete down the pipe is achieved by suspending the plug from a wire. Different types of plugs are shown in Fig. 17.6. The rod and plate plug is generally the most used. In the rod and plate plug, the length of the rod should be approximately 5 times the diameter of the plate. The plug should be passed down the pipe, prior to pouring, in order to check that there is sufficient clearance. Plugs made of rubber balls should not be used since the water pressure will decrease the ball diameter and therefore not prevent the water and concrete from mixing. Figure 17.7 shows the detail of the rod and plate sections of the tremie pipes.

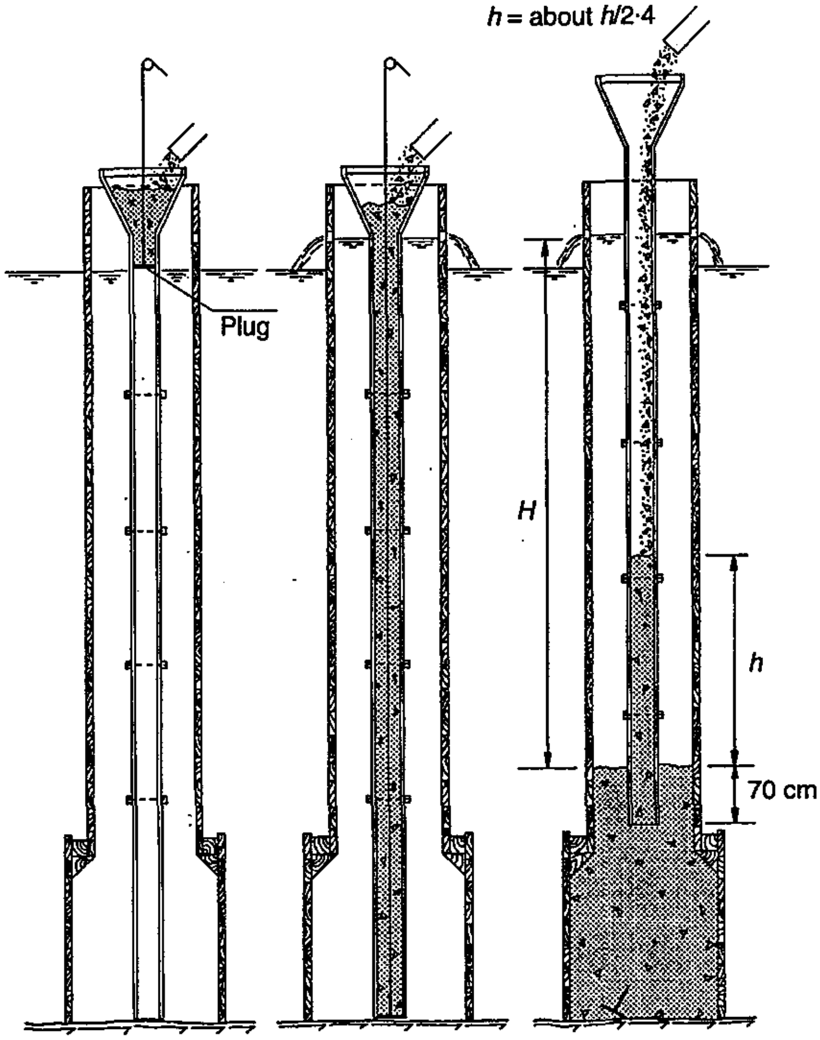

Fig. 17.5. Start of the tremie pipe method

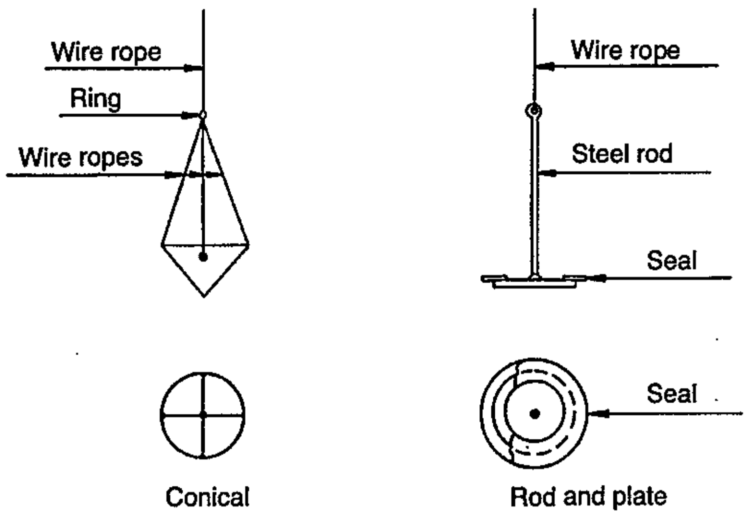

Fig. 17.6. Different types of plugs

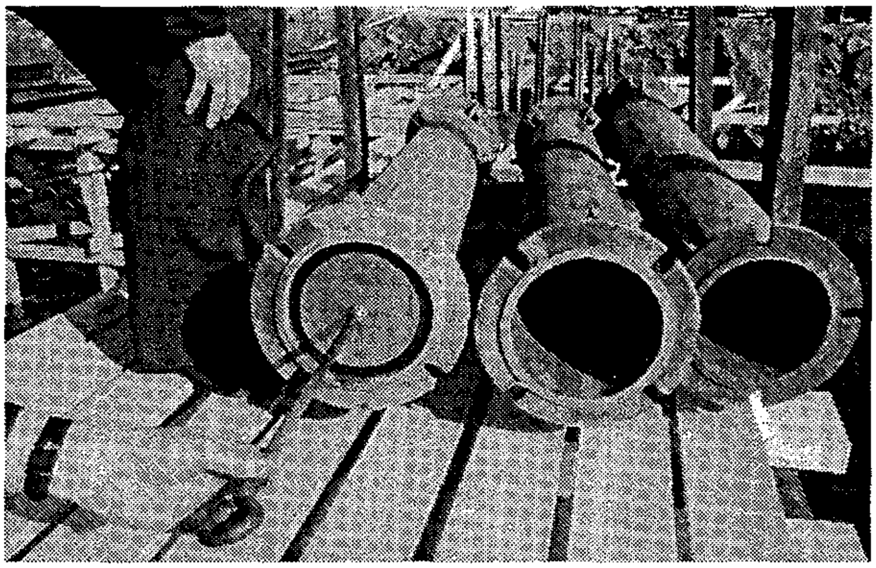

Fig. 17.7. Detail of rod and plate

The first concrete batch should always be an oversanded and cement-rich mix. The plug is then slowly lowered down the pipe, while the pipe is continuously being filled with concrete. When the plug reaches the bottom of the pipe the pipe should be filled to the top. The tremie hopper should also be filled completely and additional concrete should be kept ready in a hopper above the tremie hopper. The wire should then be cut and the pipe should be lifted slowly, whereupon the concrete will start flowing into the formwork. The pipe may then be lowered to reduce the speed of the concrete out of the pipe. With constant refilling of the pipe, the concrete will be pushed upwards and outwards in the formwork, as shown in Fig. 17.8.

The concrete will flow from the pipe into the poured mass of concrete, as shown in principle in Fig. 17.8. The concrete moves down the pipe and will flow the easiest way after leaving the pipe, i.e. flow up along the outside of the pipe due to the friction of the reinforcement to the surface of the concrete and roll over to the formwork, as shown in Fig. 17.9. This means that nearly all concrete will come in contact with water. It is, therefore, very important that the pipe outlet is submerged at least 70 cm into the concrete. The reason for this is to slow down the flow speed of the concrete as it comes up along the pipe and rolls out on the top of the concrete, as shown in Fig. 17.8. If the flow rate is too high, the cement in the concrete might be washed out. In this case the cement will discolour the water and one may get white foam floating to the top of the water. To obtain successful underwater concrete pouring, the concrete must have a correct flow both in the pipe and in the formwork.

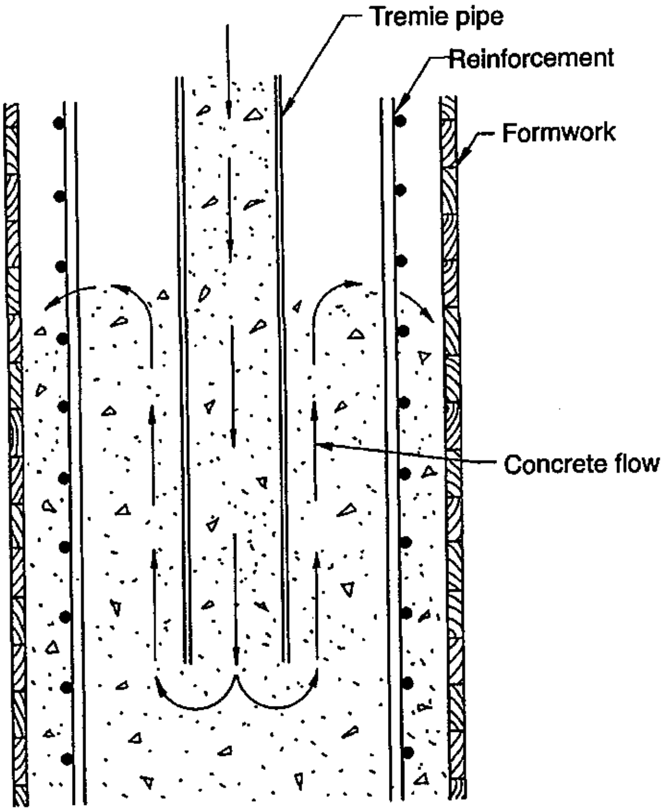

Fig. 17.8. The concrete flow from the pipe into the column

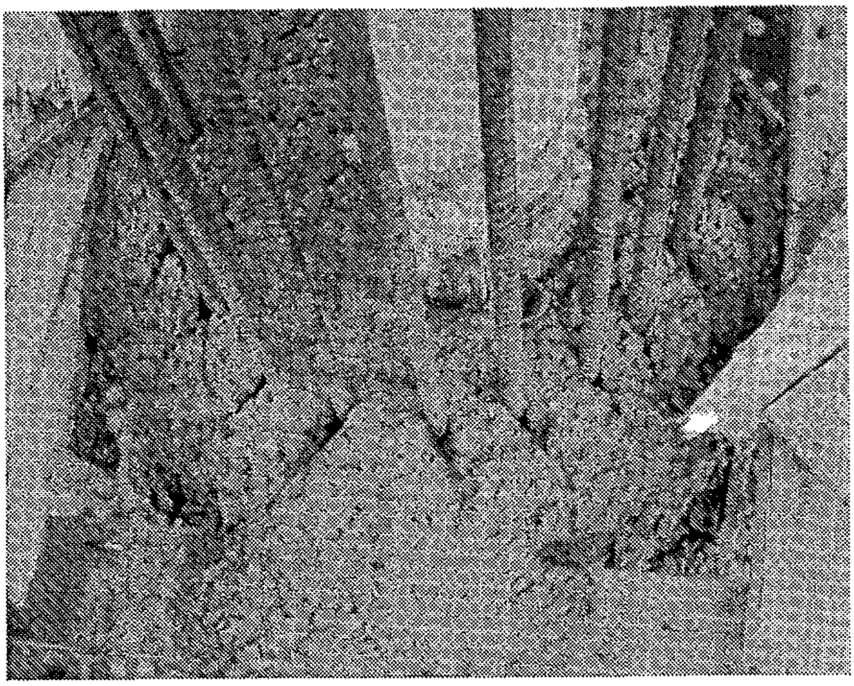

Fig. 17.9. The concrete flow from the tremie pipe

During the pouring, the pipe outlet should be submerged at least 70 cm into the concrete. If the immersion depth is too small, a breakthrough of water from the outside into the pipe can occur, or the concrete flow up along the tremie pipe up to the surface can be too fast with the result that the cement is washed out. If the immersion depth is too large the concreting speed is reduced to virtually zero. A main advantage by using a concrete pump for underwater concreting is that the immersion depth can be kept very safe without suffering from reduced casting speed.

Accurate measurements must be made all the time to check that the immersion depth is kept at a safe level as the concrete rises in the formwork and the pipe is withdrawn upwards. The difference in level between the concrete surfaces inside and outside the pipe is usually about $\frac{1}{4}$ of the water depth in the formwork, as shown in Fig. 17.5. The following two levels must be continuously checked:

(a) outlet of tremie pipe
(b) concrete surface in the formwork.

When the concrete levels approach the water surface a reasonable overpressure in the supply pipe requires that a 'tower' has been built, see Fig. 17.4. If concrete pumping is applied, no 'tower' is necessary.

If the ground or sea bottom is loose or soft below the pipe, for instance clay or sand, a metal sheet, a concrete tile or the like is laid under the outlet of the pipe to avoid disturbance of the loose material.

When the underwater concreting has started, the concreting should and must proceed continuously without breaks or other interruptions until a predetermined level above the water level is reached.

Generally, concreting underwater should proceed as quickly as the formwork pressure allows. For columns with a cross-section of about 0.5–2 m^2, a concrete filling rate of about 2–4 m of column length/h is usual depending on the strength of the formwork. For walls and structures with larger cross-sections the filling rate may be reduced to about 60 cm/h. If for any reason the filling rate falls below 60 cm/h, the flow of concrete is normally unsatisfactory. If the pipe outlet is embedded too deep into the concrete, or the filling rate has been too low, a 'plug' may build up in the pipe. By the use of a rod or a special vibrator inside the pipe, the concrete flow may be restarted without letting water into the pipe, which may happen as a result of lifting the pipe too high. Again a concrete pump shows advantages, as plugs which are troublesome for ordinary tremie, are no problem to a pump. Vibration of the concrete should generally not be allowed in the formwork itself due to the risk of washing out the cement.

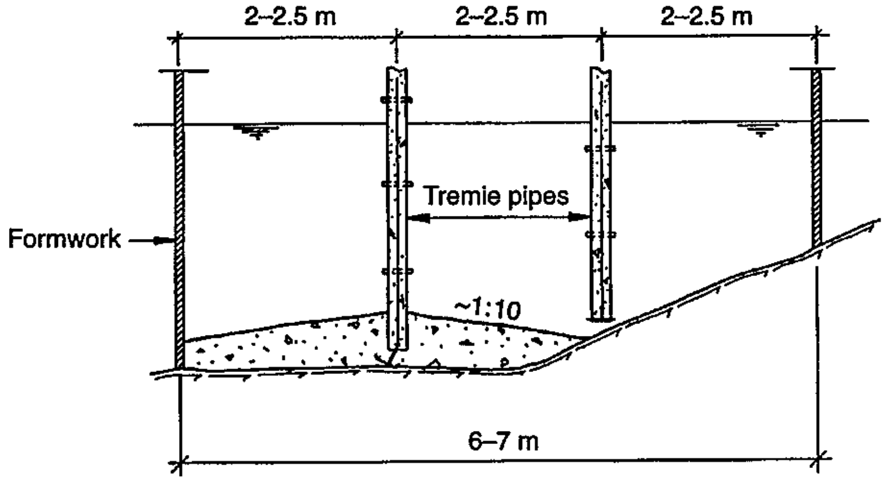

Fig. 17.10. Tremie pipes on a sloping bottom

For more extensive works, where an interruption has far-reaching consequences, the pouring speed may in extraordinary cases, be temporarily reduced. If the pouring speed is reduced trouble is not usually experienced immediately. The trouble with poor concrete flow is usually experienced after 2–6 h. The pouring must be kept at a high speed, and the concrete, the temperature and other relevant conditions must be met to suit underwater concreting. In addition, qualified personnel must be present on the site. In such cases, testing and sampling, in order to document that the structures are satisfactorily poured, must be undertaken.

When the area to be poured is large and the bottom is uneven and sloping with depressions, the pouring should start in the pipe, which is deepest when several pipes are used. The remaining pipes are successively brought into use when their outlets are about to be covered by concrete poured from the lower pipes. Just prior to reaching these levels, the corresponding pipe must be filled with concrete, as shown in Fig. 17.10.

When new concrete is poured from the hopper, a characteristic hollow thump is heard in the pipe. Shock feeding of the pipe must be avoided as this may lead to pockets of air in the pipe. To a certain extent air locks may be prevented by hanging plastic hoses or a ventilation tube down through the pipe, as shown in Fig. 17.11. The concrete should always be placed on the tremie hopper wall and not directly into the pipe.

Figure 17.12 shows how the concrete will flow during concreting of an underwater slab, and what can happen if the pipe, by mistake, is lifted up too much. The concrete will flow to the surface at too high a speed resulting in a 'blow out' and in washed out concrete. As is

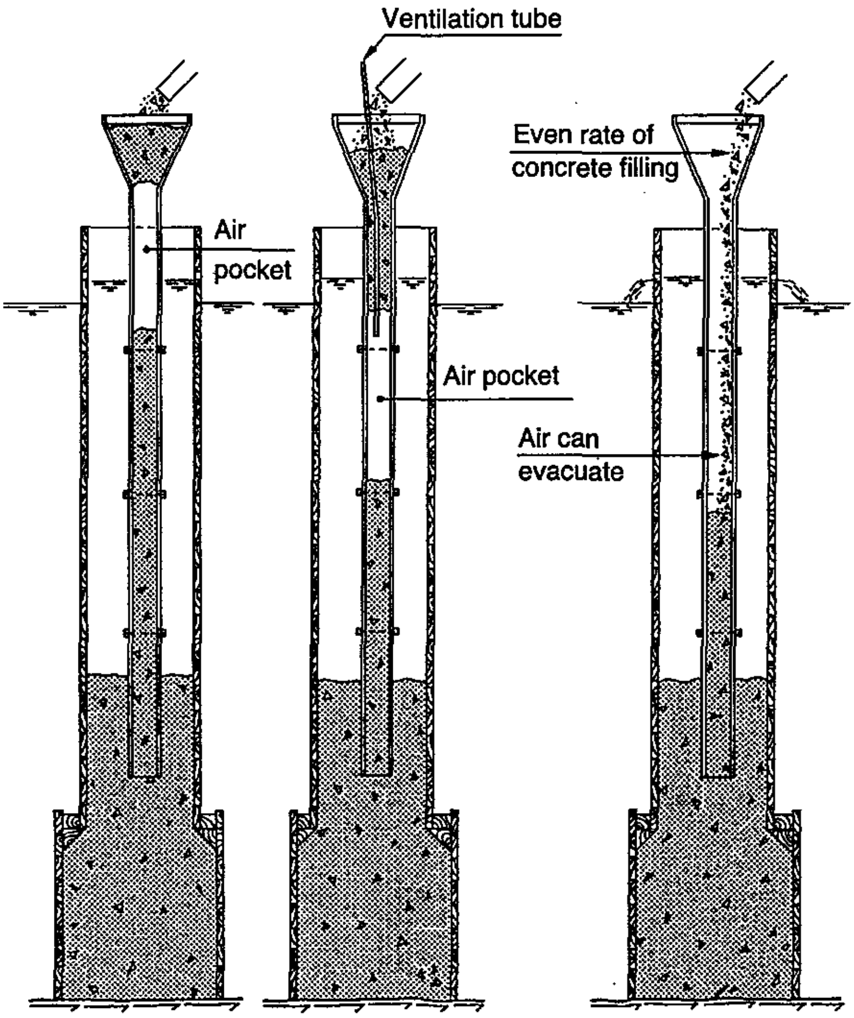

Fig. 17.11. Air pocket in the tremie pipe

also shown from the figure, the water due to the water pressure can penetrate into the pipe with the result of washed out concrete.

Completion of a foundation underwater can be done as shown in Fig. 17.13. When the pouring is nearly completed, the concrete will stand in the tremie pipe at an equilibrium height as shown. The pipe is then slowly filled with water to reduce the possibility of a breakthrough of water from the outside through the concrete and into the pipe. The pipe is then slowly withdrawn from the foundation. When a concrete structure has to be levelled off underwater, experience has shown that the tremie pipe method will give a good result when done in a careful way. The concrete is poured some centimetres higher than the required level. Then before the concrete has set and hardened the excess concrete is removed by scraping in one direction across the whole surface at once.

If a sound surface with the greatest possible compressive strength is required, the upper 2–3 cm of the finished surface must be removed and cleaned by water jetting when the concrete has hardened.

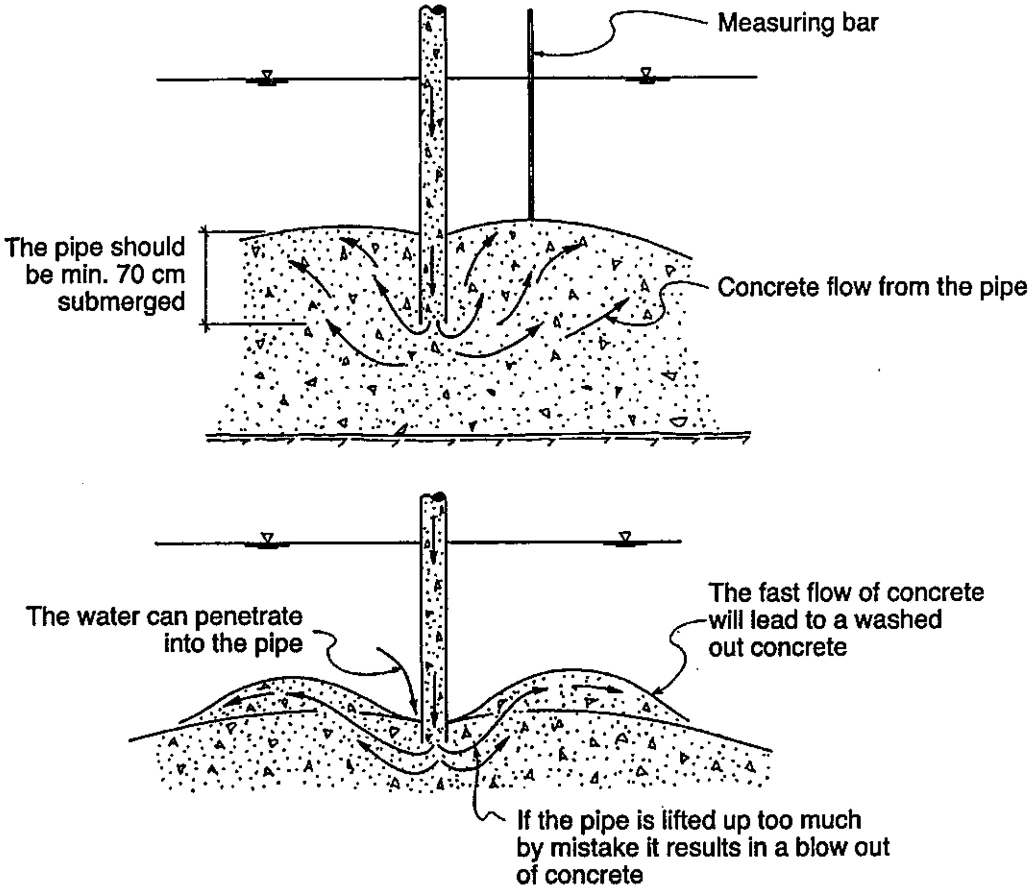

Fig. 17.12. Concreting of an underwater slab

When concreting temporary structures, a larger area may be covered by carefully moving the tremie pipe sideways. This should not be allowed in the case of permanent structures as washing out of cement cannot be avoided.

17.3.5 Structural aspects

Design and construction of underwater concrete structures should be in accordance with accepted international codes and regulations. This chapter is based on common and proven Norwegian practice and guidelines.

When designing reinforced concrete structures that have to be poured underwater, the method of concreting has to be taken into account during the design phase. For the tremie pipe method the least horizontal dimension of a structural cross-section is governed by the size of the tremie pipe flange, which is normally about 35 cm for a 20 cm pipe. A reinforced column must therefore be not less than 70 cm in diameter, and a reinforced wall must be not thinner

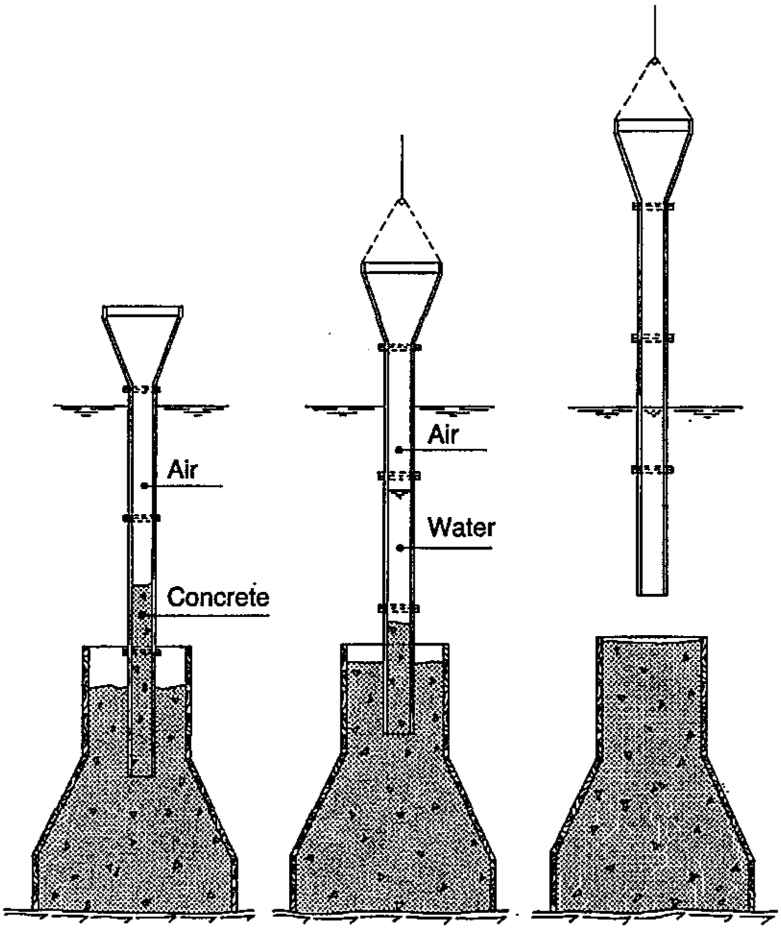

Fig. 17.13. Completion of a pouring under water

than 60 cm. In shallow water, where a pipe without flanges can be used, the minimum thickness can be reduced.

Sufficient spacing between the reinforcing bars must be provided for the tremie pipe. Figure 17.14 shows the most usual arrangement of the reinforcement stirrups in a rectangular column. The stirrups should be made of steel of not less than 10 mm diameter. Formwork and reinforcement baskets for concrete columns are often prefabricated and mounted ashore. The formwork with the reinforcement must, therefore, be sufficiently strong to be lifted into the water. From a construction

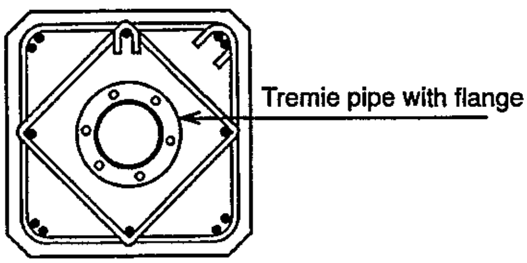

Fig. 17.14. Stirrup arrangement

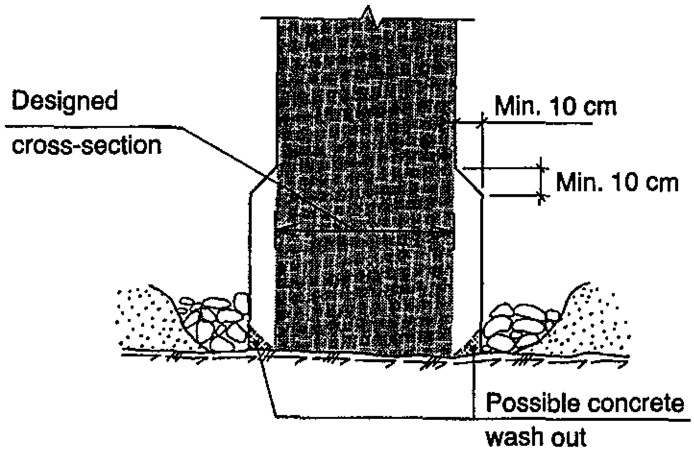

Fig. 17.15. Enlargement of column cross-section due to possible concrete washout

and maintenance point of view, a circular cross-section for the columns is best.

Where the capacity of a concrete section is fully utilized, the base should be increased in size by at least 10 cm or preferably 15 cm in all directions in order to compensate for possible concrete washed out at the start of pouring, as shown in Fig. 17.15. Reductions in column or wall cross-sections must not occur suddenly. Cross-sections should be formed in such a way that concrete may easily flow and fill the formwork. Reduction in cross-sections or other changes in structural shape should be formed with slopes of not less than 45°, preferably 60°.

When concrete is poured towards an upper horizontal surface, the possibility of a washed out layer of about 1–4 cm can occur. The top reinforcement should therefore be given an extra 5 cm of cover.

Underwater concreting of slabs by the tremie pipe method is a difficult operation. Even under favourable conditions the concrete is likely to have a slope of not less than 1 : 10. In practical terms, this will determine both the minimum thickness and the horizontal dimensions of the structure. Usually a minimum thickness of 80–100 cm of the slab represents a practical limit. If the concrete has a good flow and a concrete pump is used to obtain a high pouring rate, the minimum thickness can be reduced.

17.4 Concrete production of tremie concrete

17.4.1 General

The requirements for the concrete materials and strength should be in accordance with accepted and relevant codes and regulations. The basic concrete mix should comply with the following.

17.4.2 Cement

The most suitable cement for berth structures is ordinary Portland cement. Sulphate-resisting Portland cement needs only to be used in special cases where the sulphate concentration and/or the sea temperature is much higher than in normal Atlantic waters. The importance of a moderately C_3A content of less than 8 per cent has been clearly demonstrated by important studies of the cement properties and their effect on concrete durability.

The cementious material content should in the tidal zone (Zone 2) and the splash zone (Zone 3) normally be not less than 400 kg/m^3 of compact concrete. If it can be shown that the workability can be kept unchanged by use of filler, the cementitious quantity can be reduced to 350 kg/m^3. The cement content should be not less than 325 kg/m^3 and the content of fine aggregates less than or equal to 0.2 mm including cement should be about 400 kg/m^3.

Silica fume has performed favourably in underwater concrete. The dosage should preferably be in the range 7–10 per cent of the cement content.

17.4.3 Water

For concrete production only fresh potable water should be used. Seawater may not be used as mixing water, or be used as curing water on the young concrete.

17.4.4 W/C ratio

The water/cement ratio with Portland cement or the water/binder content where silica fume is used should not exceed 0.45.

17.4.5 Aggregates

The aggregates shall be non-reactive. The use of low alkali cements as well as blended cements will improve the resistance against alkali aggregate reactions (AAR).

The aggregates for tremie pipe concrete should be well graded with an excess of fine gravel. The total amount of aggregate passing a 0.25 mm sieve should be between 8 and 15 per cent. The maximum size of the aggregate should normally not exceed 22 mm and under no circumstances shall aggregates larger than 32 mm be used. Natural river gravel should preferably be used. If this is not available, crushed rock with a cubical shape should be used. Sea sand or sea aggregates contaminated with salt, etc., should never be used even if there is sufficient freshwater available for washing.

The aggregates for tremie pipe concrete should contain 10–25 per cent less coarse material than in normal vibrated concrete due to the flow in the tremie pipe. The percentage by weight of coarse aggregates should be about 45–48 per cent and the fine aggregate should be about 55–52 per cent. Usually about 45 per cent of coarse aggregates larger than 8 mm, and 55 per cent of fine aggregates smaller than 8 mm are used. The first batch to start the concreting should always be even sandier.

17.4.6 Workability

The workability must be adequate to ensure satisfactory placing of the concrete in the formwork. For tremie pipe concrete this will correspond to a slump of 18–22 cm.

17.4.7 Admixtures

Plasticizer should be used in underwater concrete. When pouring slabs, or where large cross-sections and low pouring rates occur, retarders should also be used. When pouring in the tidal zone where freezing temperatures can occur, air-entraining agents in addition to plasticizing agents are mandatory.

During recent years, admixtures with the property of increasing the cohesion of the fresh concrete mixture or AWO admixtures have been introduced. These admixtures, which are cement binders, give the fresh concrete a high resistance to washout and segregation during placing of concrete underwater. Since these types of admixtures make the concrete also highly flowable and self-compacting, they should be used if one has to concrete slabs underwater of a thickness of less than 80–100 cm. The disadvantage with these types of admixtures is that they make the concrete very sticky and therefore make it more difficult to clean the mixing and transport equipment compared to ordinary concrete. The concrete also moves slower in the tremie pipe.

17.4.8 Concrete compressive strength

The compressive strength for concrete underwater should at a minimum be C35 (Cube strength). The maximum prescribed strength for ordinary concrete should be C 55 and maximum C 45 for AWO concrete, see below.

Compressive strength class may vary for the different structural parts, but as a minimum a strength class C 40/50 is recommended according to the European standard EN 206. This means a minimum characteristic cylinder strength of 40 N/mm^2.

17.5 The AWO-concrete

17.5.1 General

The AWO concrete is a special underwater concrete with an anti-washout agent added, such as the Rescon T product from Norway or equivalent products. The agent is a formulation of a stabilizer, high-range water reducer (superplasticizer), special fillers and additives. The superplasticizer ensures that the cement flocks are adequately dispersed. The stabilizer encapsulates the cement grains, which prevents the cement being washed out, even when in close contact with water. This is illustrated in Fig. 17.16. In the left tube concrete with anti-washout agent added is falling freely through water without the cement being washed out. In the right tube, which has concrete without anti-washout agent added, the cement is washed out.

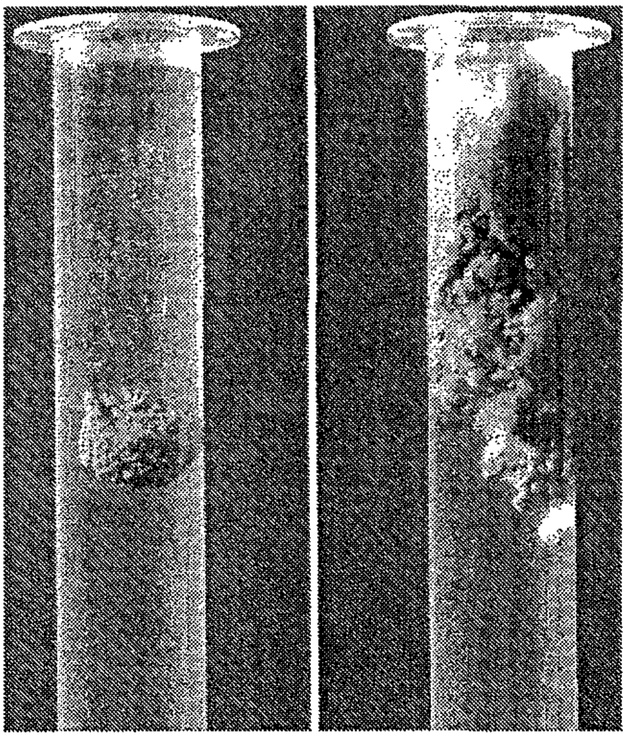

Fig. 17.16. Concrete with (left) and without (right) anti-washout agent added

When adding anti-washout agents, the concrete properties are radically altered compared to the concrete for ordinary tremie pipe concrete. Anti-washout concrete makes the diver's work more efficient; the visibility makes it possible to control and correct during casting.

With a correct mix design, the AWO concrete flows easily. The yield stress is extremely low, allowing the concrete to flow to a nearly even surface (self-levelling). It can pass obstacles, surround any reinforcement and fill the form completely. In situ core tests reveal perfect self-compacting abilities. The flow through water is relatively slow, due to its high viscosity. The AWO concrete retains its slump for a substantial period of time, increasing as the dosage increases. This allows for longer transport and casting over several hours. Adjustment of slump flow can be done on site in a sufficiently efficient truck mixer.

17.5.2 Mix design consequences

It is the dosage of admixture rather than the sand–stone ratio, or the cement content, that is decisive for the AWO concrete's water demand. The higher the amount of anti-washout agent, the more mixing water is needed. Compared to ordinary underwater concrete, an increase of 30–50 litres of water is not unusual.

As opposed to traditional underwater concrete or tremie pipe concrete, the amount of coarse aggregates must not be reduced in AWO concrete. Equal amounts of coarse (>8 mm) and fine (<8 mm) aggregates can be chosen. The aggregates should be well graded with a maximum size of 16–26 mm. Rounded particles are always preferable, but crushed stones that are not too flaky or elongated are highly acceptable. As with normal underwater concrete, the first batch with AWO concrete should be more sandy.

Dependent on casting methods and type of cement used, the addition of 5–10 per cent of condensed silica fume, by weight of cement, is advantageous. The fine particle shape, the fineness, and its pozzolanic efficiency both improves the flow and the inner cohesion in fresh concrete, as well as improving the long-term compressive strength, the permeability and the durability of the hardened concrete. The addition of silica also reduces the retarding effect of the anti-washout agent.

As in ordinary concrete, the final strength of AWO concrete is decided by the water to cement ratio, and the adding of silica contributes positively in this respect. The influence of the cement type is as for concrete in general, with the normal differences in early strength development. The anti-washout agents have a noticeable retarding

Table 17.1. Two mix designs for AWO concrete/m³

Water/cement ratio	0.42	0.50
Rescon T	15	15
Total water	220	225
CEM I — 42.5 R	450	430
Silica fume (CSF)	38	20
Sand 0–8 mm	840	850
Stones 8–22/26 mm	840	850
Superplasticizer	2–3	2–3

effect, which becomes significant at lower temperatures e.g. below 10 °C. Using cement with higher Blaine and adding silica fume reduces this retardation of setting.

Deriving from this discussion, Table 17.1 shows two examples of mix designs for AWO concrete/m^3.

17.5.3 Mixing procedure

Anti-washout agents can be added into a central mixer or directly into a truck mixer, providing this has a sufficiently efficient mixing capacity. When added into the mixer of a ready-mix plant, the addition can be done either on the aggregate scales or at the same time as the aggregates are put into the mixer, but it can also be added after the ordinary concrete is mixed. The correct amount of water should be added at once, since adjusting flow by water after the introduction of the anti-washout agent is both time consuming and extremely difficult because of the anti-washout properties. Parts of the concrete will stick to the sides of the mixer, especially if the water content is too low; thereby it is more difficult to empty the mixer.

The best method is to add the powder directly into the truck mixer. The efficiency of the mixer must be verified beforehand, but most automixers are perfectly capable of mixing AWO concrete. The addition can be done in two ways; either by adding the powder into the concrete flow as the batch is poured into the truck mixer, or by placing the powder in the truck prior to the fresh concrete. In both cases rapid rotation of the mixer is essential. Both methods require a thorough mixing at full rotation speed of the mixer. The minimum time of mixing at full speed, while the truck is at a standstill, is 15 min. During transport, the mixer is normally slowly rotating, but the general stability of the AWO concrete even makes it possible to transport the concrete without rotation.

If, on arrival at the construction site, one finds that the flow is not high enough, it is possible to adjust the flow by adding a high range water reducer/superplasticizer. Superplasticizers based on both melamine and the new co-polymers can be used. Sulphonated naphthalene must not be used. It is essential that this adjustment be done in co-operation with the diver, who is able to verify the actual flow into the form. In this way, the corrections can be communicated to the ready-mix plant to ensure the correct mixing of the following batches.

17.5.4 Casting procedures

Many problems resulting in damaged structures in ordinary underwater concrete constructions occur as a result of bad craftsmanship, lack of knowledge or bad planning. The introduction of an anti-washout agent into a sound concrete does not make these parameters superfluous. It is still essential to execute the casting according to codes and good craftsmanship, and, indeed, it is absolutely necessary to carry out a significant amount of planning prior to the concreting. If one deviates from the ideal plan, unforeseen problems can occur and must be handled accordingly. The better the planning, the more adequately these problems can be solved. A well-known saying goes like this: plan ahead, and you won't go wrong! For proper planning see Section 17.8.

17.5.5 Placing and casting methods

The AWO concrete can be cast in most ways, often reducing the need for costly and complicated rigs. Both new constructions and repair works at smaller depths have been successfully carried out with a crane and skip. If the concrete should sustain a free-fall through water, it is advisable to prescribe the maximum dosage of admixture (with e.g. Rescon T up to 25 kg or one bag/m^3), but just an introduction of a slope to reduce the direct free fall reduces the risk of washout dramatically. For smaller works it is also possible to use buckets and simply pour the concrete into the form.

When executing larger works, and with larger sea depths, the use of pumping is normal. Even with dosages as low as 10–15 kg/m^3, the anti-washout effect is high.

When pumping AWO concrete, it is essential to give the concrete time to flow slowly through the pipe. Trying to increase the speed only results in a higher pumping pressure and rising temperature in the hydraulic oil, and thus increased wear on the equipment. The high viscosity prevents fast pumping, so the only measures to be taken to increase rate of casting are either to increase the diameter of the pipes (a minimum of approximately 12.5 cm is recommended) or to use more pumps.

17.5.6 Formwork consequences

The AWO concrete has extreme flowing abilities, ensuring that the form is completely filled without any additional compaction energy needed. The active flow is also followed by a penetrating quality that enables the AWO concrete to find its way out through even apparently small holes. It is therefore paramount to ensure absolutely tight forms. The transition zone between rock/ground and formwork is especially important. A leakage here can have serious consequences.

Because of the retarding effect of the anti-washout agent, the formwork must be dimensioned to tolerate the loads of a fresh concrete pillar from bottom to top of the form. Again the zone between formwork and foundation is essential; a good anchoring either by bolting or by sand bags is absolutely necessary.

The AWO concrete is practically self-levelling if correctly designed and mixed, and it is therefore not possible to obtain slanting surfaces without an overformwork.

17.5.7 Combination

It is possible to combine AWO concrete and traditional underwater concrete in the same construction. If one can guarantee that the concrete is cast through a permanently submerged pipe, the start of the casting — that is when the concrete is most exposed to washing out — can be done with concrete into which an anti-washout agent is added. The amount of this 'start' is relative to the possible exposure to water, and slender structures with smaller cross-sections have reduced surfaces compared to massive constructions. The subsequent casting is continued with a pipe always submerged approximately 70 cm. The mix is then an ordinary well-designed underwater concrete with the appropriate water to cement ratio. The initial AWO concrete will function partly as a buffer with primary contact with water. The possibility that the normal concrete will be exposed to water cannot be ruled out, especially with complex forms and currents in the sea but, more often than not, this method has radically improved the final result.

17.5.8 Hardened concrete

For traditional underwater concrete, the reduced quality of the concrete that is in contact with water has resulted in a designed reduction of the effective cross-section of the construction. The outer 10 cm in slender constructions cannot be considered as carrying loads. For massive constructions the outer 20 cm must be excluded. In accordance with the improved performance of concrete with anti-washout agents, the Norwegian Concrete Association's publication No. 5 (*Guidelines for The Design and Construction for Underwater Concreting*) has allowed the full cross-section to be taken as load carrying. This makes for slender constructions and a reduced quantity of concrete.

Also, exposed surfaces with traditional underwater concrete are often rather porous, often leaving larger depths of 'concrete' with an extremely high water to cement ratio. With AWO concrete, practical applications show none or only very thin layers with reduced quality, thus ensuring the high quality of the concrete cover which is so essential for the durability of concrete.

All concrete follows the 'law' of slow starters: in the end the strength is higher than the 'false starters'. The AWO concrete is normally a slow starter; and the surroundings of underwater constructions can also be unfriendly in terms of temperature. On the other hand, underwater concrete always has sufficient water to ensure complete hydration of the cement. It is therefore not surprising that the development of strength, measured at in situ constructions, shows a marked growth after 28 days.

Concrete totally submerged in water does not freeze, but in tidal zones the cycles of freezing and thawing can deteriorate the concrete from within. The AWO concrete is not susceptible to air entraining agents, and is therefore not considered to be frost resistant. Reports from tests made in Sweden do show that AWO concrete resists high numbers of freezing–thawing cycles with a minimum of spalling. Nevertheless, the normal procedure is to stop using AWO concrete approximately 1–2 m below the tidal zone and then continue the concreting with air entrained frost-resistant concrete in the tidal zone and into the construction above sea level.

17.6 Damage during construction of new structures

Damage of newly poured concrete can be due to one or more of the following reasons.

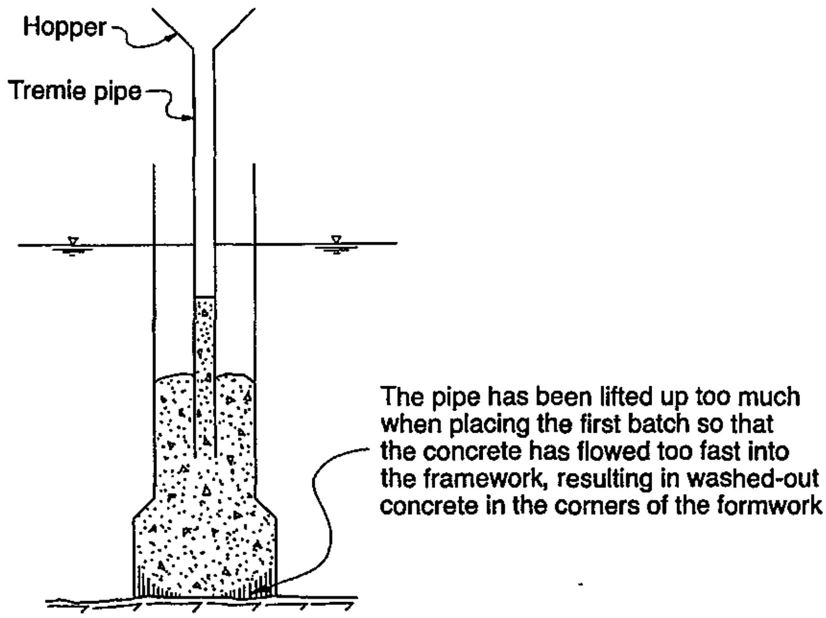

Fig. 17.17. The tremie pipe in a too high position

17.6.1 Unskilled labour

Perhaps the most common reason for damage during construction is the lack of skill and experience among the concrete workers. For instance, when starting the pouring of concrete through the immersed tremie pipe, a common error is that the lower end of the pipe is placed too high, or lifted too high, resulting in a washing out of cement and fine aggregates, as illustrated in Fig. 17.17. Similarly during the concreting of columns, the tremie pipe has been lifted out of the concrete and then been put down into the concrete again, resulting in a layer of washed out concrete, as illustrated in Fig. 17.18. Therefore a person experienced in underwater concreting work should always be in attendance.

17.6.2 Unsatisfactory concreting equipment

The capacity of rigs and other equipment is either insufficient to obtain the necessary working continuity, or otherwise unsatisfactorily adapted to the work in question.

For instance, the jointing material between the section of the tremie pipes is not tight, resulting in leakage and washed-out concrete, or the diameter of the tubes is too small causing discontinuous pouring, or the hopper has been over-filled with concrete so that some of the concrete has flowed over the hopper edge and fallen down freely through

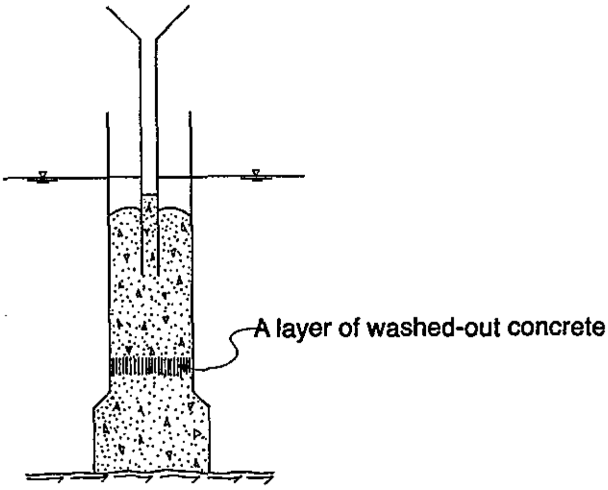

Fig. 17.18. Washed-out concrete layer in the column

the water and placed itself on top of the already-placed concrete, as illustrated in Fig. 17.19.

17.6.3 Deficient delivery of concrete

Continuous delivery of concrete of the prescribed consistency has not been planned or arranged in advance. A trial concreting will show

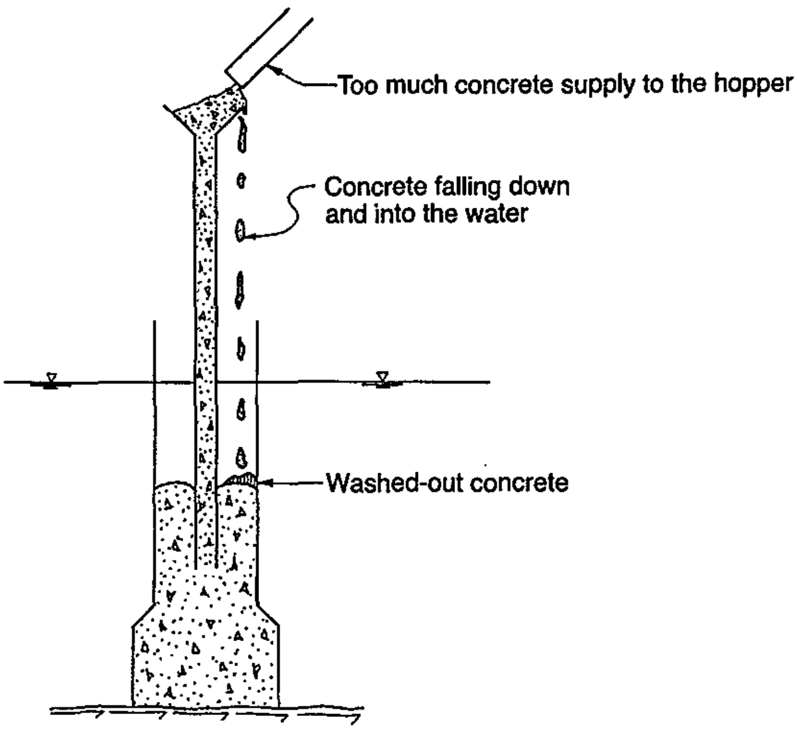

Fig. 17.19. The hopper is over-filled with concrete

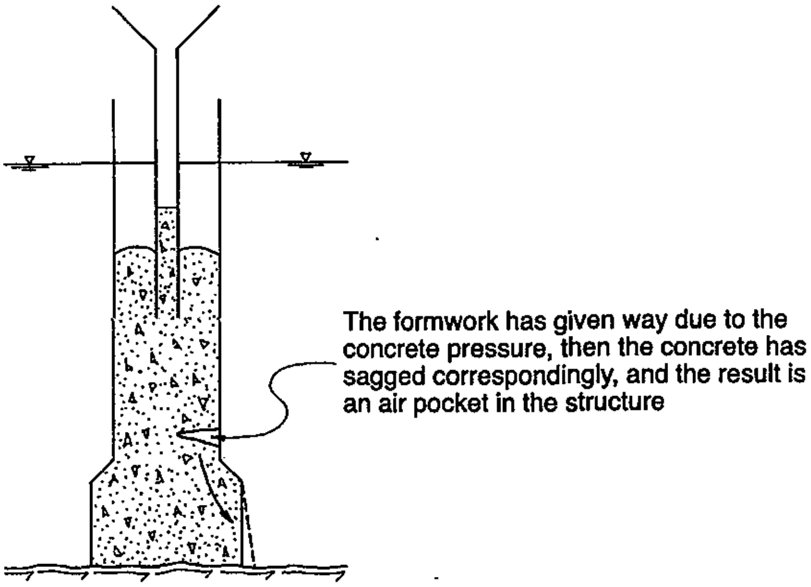

Fig. 17.20. The formwork has given way

whether, for instance, the amount of retarding agent used/m^3 concrete permits a satisfactory form filling rate.

17.6.4 Faulty formwork

Incorrect design of the formwork has caused distortion under the pressure of wet concrete, Fig. 17.20, or the formwork does not fit the underlying rock surface so that some of the concrete leaks out, Fig. 17.21, or waves

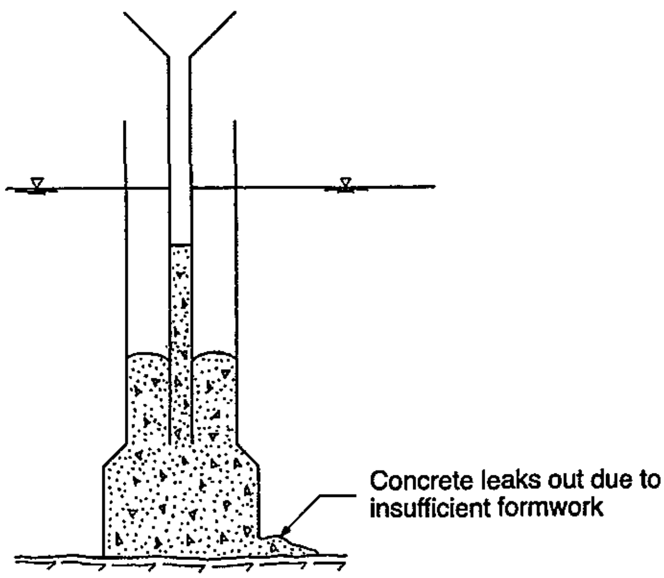

Fig. 17.21. Concrete leaks out

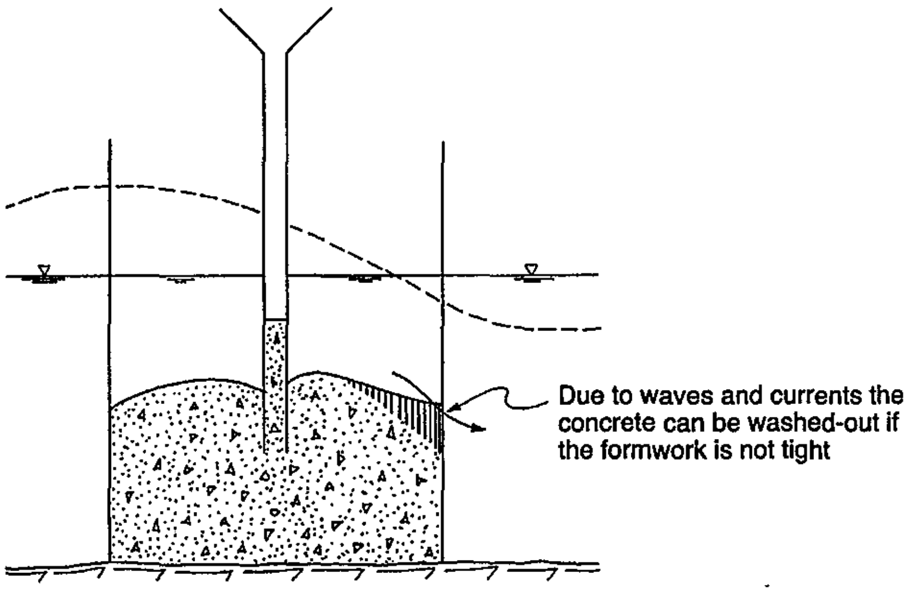

Fig. 17.22. Leaky formwork

and current wash out the concrete or especially the cement in the concrete where the form is not tight, as shown in Fig. 17.22.

17.6.5 Physical damage

Damage caused by severe impacts from ships, waves, ice, etc. are usually outside the owner's control, whereas damage from overloading, e.g. the formwork during construction due to application of too high live loads or construction cranes, etc., could be avoided.

17.7 Repairs of new concrete

Repairs on new concrete mean the repair of or immediately after the construction of the new concrete has finished. The supervising engineer must have sufficient knowledge of concrete technology, theory of structures, construction of underwater concrete structures and construction supervision. For instance, if anything irregular happens during the pouring of the concrete, the supervisor must be sufficiently competent to understand what the consequences can be and what measures must immediately be taken.

The supervising engineer should therefore be properly informed in advance about the qualities required in the finished structure, so that he will be able to judge whether to:

(a) Stop the pouring and remove at once all the concrete already poured. For instance, if the tremie pipe during concreting of an important column is lifted into a too high position so that water penetrates into the tube, the formwork shall be removed, the concrete washed away with high-pressure water, and the work started anew.

(b) Stop the pouring and continue the next day. This could be the correct measure if, for instance, the concreting of the column described above was nearly finished when the tremie pipe was lifted too high. The formwork could then be removed above the concrete surface and the washed-out concrete on top (at least 10 cm) be removed by divers. Concreting may be continued after at least 12 h. In most cases, the joint between the two parts of the column has to be strengthened. A reinforced concrete mantle surrounding the column then increases the column cross-section at the joint.

(c) Continue the pouring of concrete and make the repair afterwards. For instance, during the concreting of a long, high and thick concrete wall, using four or five tremie pipes at the same time, one of the pipes has been lifted too high so that a fault occurs. In less important structural elements, it can then be permitted to put the pipe down again and continue the pouring, provided the contractor finds it more convenient to make a repair when the wall is finished. In such cases, the contractor shall cover all costs of repair and control including a diamond drill test.

17.8 Concrete plant and supervision

17.8.1 General

As already mentioned, during the pouring of underwater concrete it is important that the work proceeds continuously without any breaks. The contractor must, therefore, carefully plan and organize the concrete work. The batching plant and the transportation system has to be dependable and must have sufficient capacity. For larger or important jobs an additional batching plant and a power generator must always be provided. The additional equipment must be capable of starting at short notice to avoid any interruption in the pouring of the concrete.

17.8.2 Construction supervision

Both the Consulting Engineer and the contractor should document that they have previous experience of underwater concrete work before they take or are given any responsibility for underwater concrete work.

Before the concreting starts, a check of the contractor's equipment and personnel should be made to ensure that all requirements are fulfilled.

17.8.3 Checklist for underwater concreting

The following checklist for quality control of underwater concreting with the tremie pipe method is strongly recommended by the Norwegian Concrete Association.

Planning of the concreting operations

(a) Briefing of key personnel like inspectors, foremen and representatives of the contractor and others.
(b) Check that the concrete mix design is executed and that the mix is approved.
(c) Check that the pouring rig/plant is sufficiently designed.
(d) Check that the owner and the designer approve possible deviations from the specifications or regulations.
(e) If requested, test pours in order to check the workability of the concrete or the suitability of unusual rig arrangements.
(f) Pouring speed, submerged pipe length, sectioning of the area has to be planned.
(g) Sufficient divers for inspection and work operations
(h) Has the contractor approved the specified method, or does he want alterations concerning equipment or methods?
(i) Consider possible tidal variations.

Before pouring; concrete delivery

The aggregates:

(a) Sieve analysis of sand and gravel.
(b) Grain shape (for good workability and reduced segregation).
(c) Impurities.
(d) Maximum size of aggregates.

The admixtures and their effects:

(a) Air entraining agents, specified air content.
(b) Plasticizing agents, specified workability.
(c) Retarding agents, specified retardation.
(d) Sufficient amounts of admixtures available.

Cement:

(a) Sufficient amount of specified cement available.
(b) Prescribed type.
(c) Hard lumps.

Batching plant:

(a) Sufficient capacity for continuous pouring.
(b) Scales in order.
(c) Reliable water supply and proportioning.

Transport arrangements:

(a) Sufficient capacity.
(b) Reliability (weather changes, etc.).
(c) Flexibility.

Transport distance (danger of segregation, etc).
Sufficient capacity for filling the pipes when starting.
Plan of work for continuous pouring (no breaks, i.e. lunch).
Men available, shift work.
Additional stand-by equipment.

(a) Sufficient capacity.
(b) Operational.

At the site
Foundation:

(a) Check cleaning.
(b) The base shall be free from mud, fines, seaweed, etc.
(c) Centre bolt shall be removed.

Formwork:

(a) Is shape suitable, is good outlet of the concrete secured?
(b) Sealing of formwork and footing.
(c) Sufficient strength?
(d) Clean and free from pieces of wood and other debris.

(e) Dimensions in accordance with drawings, specifications, sufficient tie rods and/or anchors?
(f) Displacement of the formwork.
(g) Openings for water outlet at the top.
(h) Are horizontal surfaces avoided as far as possible?
(i) At changes in cross-sections.

Reinforcement:

(a) Correct placing.
(b) Sufficiently tied and stiffened.
(c) Cover.

Tremie pipe:

(a) Check dimensions, strength, waterproof joints and valves.
(b) Check hose quality for pumped concrete.
(c) Check of tremie pipe position in the mould, distance from the mould and area to be covered.
(d) Check pipe construction, the hopper and lifting arrangements.

Checks during concreting operations
Check that start-up is done according to plan:

(a) Plug and plug suspension.
(b) Hoses for venting the tremie pipe.
(c) Protection against concrete overflow at the hopper.
(d) Start pouring in deepest pipe.

Quality control of concrete:

(a) Receipt of dispatch notes with indications of time.
(b) Concrete, that shows signs of separation, shall be rejected.
(c) Check slump.
(d) Check air content.
(e) Check compressive strength.

Ensure that filling of concrete into the tremie pipe is controlled by plug.
Check pipe is submerged by measurement.
Check that the height of the concrete inside and outside the tremie pipe is held above the critical value.
Check that no water enters the tremie pipe.

On restarting ensure that proper cleaning of mud and starting procedure are performed.
Check that the reinforcement does not move upwards. Ensure that there is sufficient over-height of concrete when pouring is completed.
Ensure that lateral displacement of pipes does not occur.
Ensure that pouring proceeds smoothly from all pipes.

After concreting

Check the removal of the upper layer (laitance) after the concrete has set.
Ensure thorough cleaning of pipes, etc. for later use.
After removal of formwork test the surface with a sharp object.
Core drilling must be done in doubtful areas.
Make a report.

References and further reading

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18

Concrete deterioration

18.1 General

Deterioration of concrete in marine environments can occur in different zones. The usual horizontal zonal division of structures in marine environments is as follows and is shown in Fig. 18.1.

(a) Zone 1: the submerged zone, which is the area below LAT, i.e. the part of the structure that is always submerged in water.
(b) Zone 2: the tidal zone, which is the area between LAT and HAT.
(c) Zone 3: the splash zone or the area above HAT, which is periodically exposed to water from waves. Berth beams and bottoms of berth decks are normally included in this zone.
(d) Zone 4: the atmospheric zone or the areas which are only sporadically exposed to seawater due to splash from waves and spray from wind. Tops of berth decks, concrete walls on beaches, etc. are included in this zone.

These four zones can have different requirements on the composition of the concrete, the placing and covering of the reinforcement, the designed load coefficients, the materials coefficients, etc.

Experience has shown that any defect or weakness in a concrete structure will show up relatively quickly in a marine environment. It is therefore very important that anyone who designs structures for marine environments has a thorough knowledge of the potentially destructive mechanisms endangering the structures and how to repair the structure.

The reasons for deterioration of concrete could be that the design engineer may have chosen unfavourable dimensions of the structural

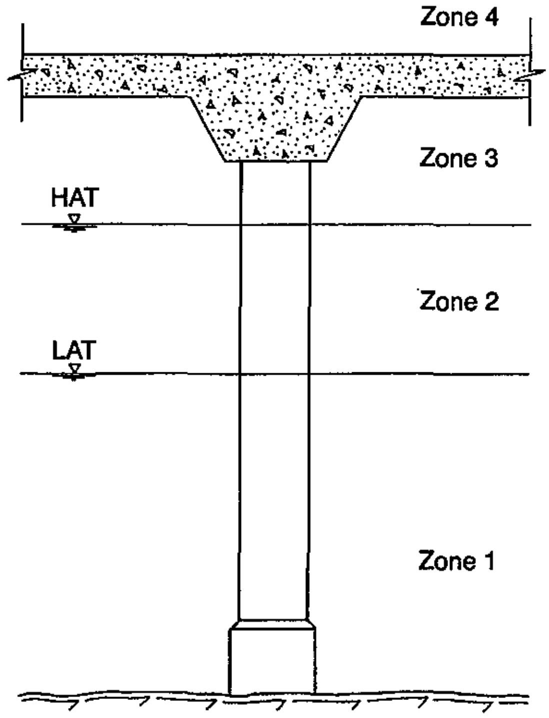

Fig. 18.1. Zonal division

elements by prescribing, for instance, too high and narrow beams under the berth deck, incorrect cover to the reinforcement and unfavourable locations of casting joints, or that the contractor has not carried out a satisfactory concreting. It is important that the form has been cleaned of all debris before concreting and that during concreting the reinforcement is not trodden down, leaving it insufficiently covered by concrete in the finished structure.

Likely reasons for the damage or deterioration of the concrete structures in a marine environment are:

(a) Poor quality of the concrete used.
(b) The concrete has been poured without proper care.
(c) The cover of the reinforcement bars has been too small.
(d) The surface drainage system has not been effective.
(e) There has been no maintenance or service inspection.

Under these circumstances corrosion of the reinforcement steel may start very early and go on unhindered, leading to the concrete spalling off, further corrosion and rapid breakdown of the structure. Once corrosion starts, it becomes progressively worse as the rust or the corrosion products spall and crack the concrete thus admitting more oxygen and chlorides from seawater to the reinforcement.

Table 18.1. Deterioration to be expected in the different zones

Zone	Deterioration caused by	Deterioration occurring immediately	Deterioration occurring after some years
Zone 1	Faulty formwork	×	
	Faulty pouring	×	×
	Corrosion		×
	Chemical reactions		×
	Erosion		×
Zone 2	Faulty pouring	×	×
	Freezing and thawing		×
	Physical actions	×	×
	Corrosion		×
	Chemical reactions		×
	Erosion		×
Zone3	Freezing and thawing		×
	Corrosion		×
	Chemical reactions		×
Zone 4	Corrosion		×
	Chemical reactions		×

A Swedish investigation, printed in *Väg-och vattenbyggaren* No. 9, 1986, on the causes of deteriorations of berth structures, shows that the frequency of damages can be divided into the following:

(a) Environmental conditions (frost, corrosion, salt, ice, etc.), 45 per cent.
(b) Excessive loading (ship collision, too heavy live load, etc.), 20 per cent.
(c) Wrong design of the structure, 20 per cent.
(d) Various other mistakes, 15 per cent.

Deterioration can generally be divided into the following two groups, as indicated in Table 18.1, those appearing during and immediately after pouring of the concrete, and those arising first after some years.

18.2 Durability of concrete berth structures

For concrete berth structures along the Norwegian coastline, extensive field investigations and research works have been carried out on durability and long-term performance of concrete structures in the marine environment. These works have revealed that an uncontrolled rate of chloride penetration and corrosion of embedded steel have created a serious threat to the safety and economy of the berth structures.

The research works have shown that the minimum durability requirements in current concrete codes may not be satisfactory to ensure good long-term performance of concrete structures in the marine environment. The experiences have shown that a high chloride penetration may be reached already at an early age during concrete construction before the concrete has gained sufficient maturity.

This may especially be true if the concrete construction work is carried out under rough and cold weather where the curing conditions during concrete construction can be poor and make the concrete more vulnerable to early chloride exposure compared to that under milder climatic conditions. Therefore experiences have shown that concrete in marine environments will show defects and deterioration relatively quickly if the composition of the concrete and the execution of the concreting work have been deficient. The deterioration of concrete in an aggressive or exposed environment can be due to the following.

18.3 Freezing and thawing

In concrete exposed to repeated cycles of freezing and thawing, for instance in the tidal zone, a suitable amount of air-entraining agent should be added. It is the smallest air pores of less than about 300 μm (0.3 mm) that determine the degree of resistance. Low water/cement ratio also improves the freezing and thawing resistant.

The various air-entraining agents can give a different number and distribution of air pores, and it is not possible to predict whether a sufficient number of the small pores will be achieved without trial mixing and testing in advance of the actual pouring.

18.4 Erosion

Probably the most usual type of deterioration in Zone 2 (the tidal zone) is mechanical erosion caused by wave, current, ice action, etc. Unprotected concrete columns are too often found to have the characteristic shape of an hourglass with rusted-off reinforcement, as shown in Fig. 11.15 in Chapter 11. Therefore concrete columns, seawalls, etc., that are subjected to severe erosion should be provided with up to 300 mm of cover to the reinforcement to obtain adequate life.

Due to the mechanical erosion caused by wave, ice action, etc., the minimum concrete strength should be 45 MPa. Adding reinforcing fibres to the concrete mix can substantially enhance the concrete resistance. Fibre reinforcement or fibremesh will provide an internal restraining mechanism, which will stabilize the intrinsic stresses, particularly during the first week when the concrete is most vulnerable to cracking due to shrinkage.

18.5 Chemical deterioration

Research has shown that the less resistant a type of cement is to chemical salt-water aggression, the more important it is that the permeability of the concrete is low. Standard Portland cement concrete can show a satisfactory resistance to salt-water attacks if made sufficiently impermeable.

Chemical attack by sulphates in the seawater on the calcium hydroxide ($Ca(OH)_2$) or the tricalcium aluminate hydrate (C_3A) components of the hardened cement can result in softening or disruption of the concrete. This problem is generally less severe in marine conditions than in sulphate-bearing groundwater.

18.6 Corrosion of reinforcement

Although deteriorating processes such as alkali–aggregate reaction, freezing and thawing and chemical seawater attack may also represent a potential problem to the durability and have to be properly addressed, extensive experience has shown that it is the chloride penetration and corrosion of embedded steel that represents the major challenge to the durability and service life of concrete structures in a marine environment. The corrosion of reinforcement is usually the most serious problem related to the durability and safety of concrete structures in marine environments, particularly in Zones 2 and 3. Generally the Portland cement concrete is sufficiently alkaline (basic) to initially protect the reinforcing steel embedded in concrete, i.e. a thin passivating film of gammaferro-oxide is formed on the steel surface.

Even for what is stated as 'high-performance concrete' in the marine environment, current experience has shown that for Portland cement-types of concrete, it appears to be just a question of time before detrimental amounts of chloride will reach embedded steel. For cold and rough-weather conditions during concrete construction, a rapid chloride penetration may also take place during concrete construction before the concrete has had sufficient curing and reached sufficient maturity. As soon as the chlorides have reached the embedded steel,

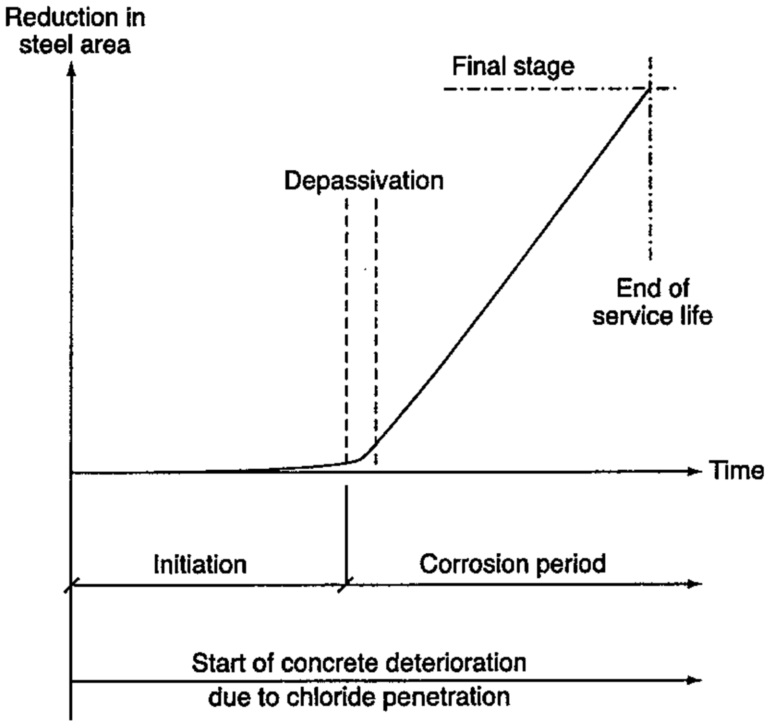

Fig. 18.2. The corrosion processes in marine environment

it becomes both technically difficult and very expensive to get the corrosion of the embedded steel under control.

The passivity of the steel is broken as soon as the pH-value of the concrete component nearest to the steel surface is reduced due to carbonation, which involves interaction with atmospheric carbon dioxide (CO_2), or as soon as the concrete is polluted by chlorides (salts) from the seawater penetrating to the steel surface. A Portland cement concrete usually has a pH-value above 13.0, whereas the pH-value for salt water is about 8.

When the chloride ions in seawater penetrate the concrete cover, the steel passivity resulting from a high alkaline concrete environment will be broken down. With sufficient oxygen and water present, this breakdown will cause a difference in the electrode potential between the exposed steel at the point at which depassivation has occurred in the oxide coated steel. The process is indicated in Fig. 18.2.

Generally the service life of a concrete structure in a chloride-containing environment can be divided into two phases, as indicated in Fig. 18.2 to distinguish these stages:

(a) The initation period is the period when the reinforcement embedded in the concrete still remains passive, but environmental changes are taking place that may terminate the passivity. The

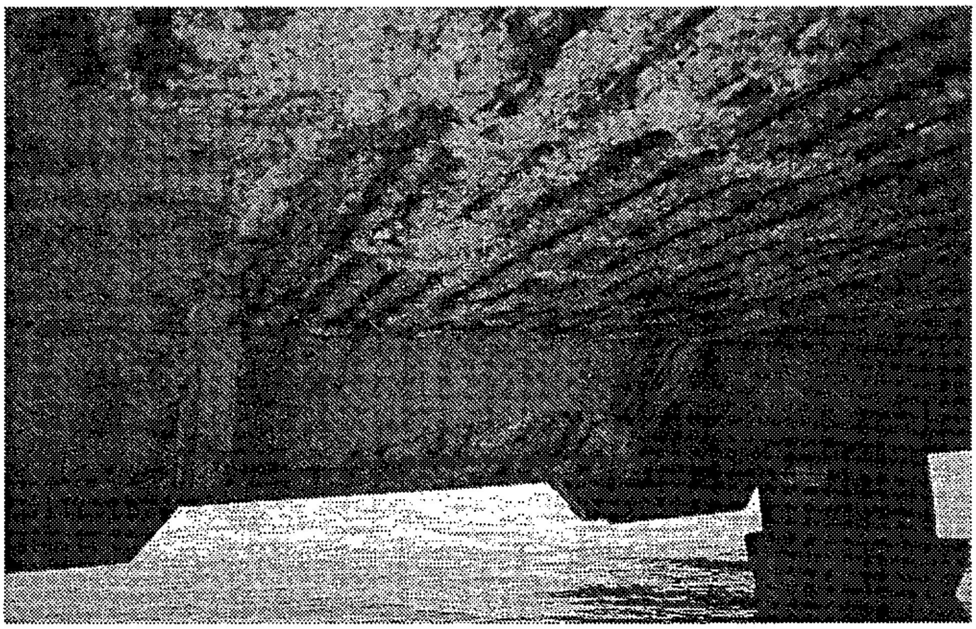

Fig. 18.3. Spalling of concrete cover due to corrosion

initiation period is the time it takes for the detrimental amounts of chlorides to penetrate the concrete cover and depassivate the embedded steel.

(b) The corrosion period is the period, that begins at the moment of depassivation and involves the propagation or starting of corrosion of the reinforcement at a significant rate until a final stage and end of the service life is reached when the structure is no longer considered acceptable on grounds of structural integrity, appearance or serviceability. The propagation period is characterized by the development of electrochemical corrosion basically controlled by the availability of oxygen, electrical resistivity and temperature. Experience has shown that the initiation period can be as short as 5–10 years.

The difference in potential provides the basis for an electrolytic reaction between the exposed steel anode and the passivated steel cathode, resulting in the steel reinforcement corroding and starting an expansive reaction which will generate sufficient tensile stresses to crack and spall the concrete cover, as shown in Fig. 18.3. These cracks can further provide easy access for oxygen, moisture and chlorides. For concrete structures that are generally well below water level and thus continuously submerged, practically no corrosion problems exist.

When the steel becomes depassivated by a chloride penetration in the area above water, the rate of corrosion will primarily be controlled by the electrical resistivity of the concrete in combination with the geometry and location of the anodic and cathodic areas forming on the steel surface of the reinforcement system. For a concrete with a given w/c ratio and maturity, the electrical resistivity is primarily controlled by type of binder, the degree of water saturation and the temperature. For a concrete based on a binder with blast furnace slag or pozzolanic materials, such as fly-ash or silica fume, the electrical resistivity is much higher than that of a pure Portland cement and, hence, the rate of corrosion becomes much lower.

To break down the passivity of the steel, fully or partly, means that the electro-chemical potential becomes more negative locally (anodic areas), while other areas of the steel surface where the passivity is still intact can promote the entry potential of oxygen and form cathodic areas. Therefore, since wet concrete is a good electrolytic conductor, a complicated system of galvanic cells can come into existence in the concrete structure. Research indicates that when concrete structures pass through several environmental zones, the concrete in the splash and tidal zones, where there is a plentiful oxygen supply, can act as a cathode for corrosion underwater. The intensity of the electromotoric force in such a cell depends upon the pH-value and the chloride concentration in the water component nearest to the steel surface and upon the amount of dissolved oxygen penetrating the concrete cover.

Experiences have indicated that during the propagation period it is very difficult to get the chloride-induced corrosion under control. When sufficient amounts of chlorides have penetrated the concrete cover, a cathodic protection system is probably the only repair method that can, in principle, stop the corrosion.

18.7 Condition survey

Physical testing should be carried out on suitable representative components and locations both at the site and in the laboratory. The test programme should include a cover depth survey of the concrete cover to the steel reinforcement, half cell mapping to determine the steel potentials and contouring, as well as chloride profiles.

To determine the amount of **chloride in the concrete** is a very important part of the survey to assess the condition of deteriorating concrete. To chemical test for determining the chloride penetration, a small sample of the concrete in the form of small diameter cores and/or dust from drillings should be collected from the berth structure

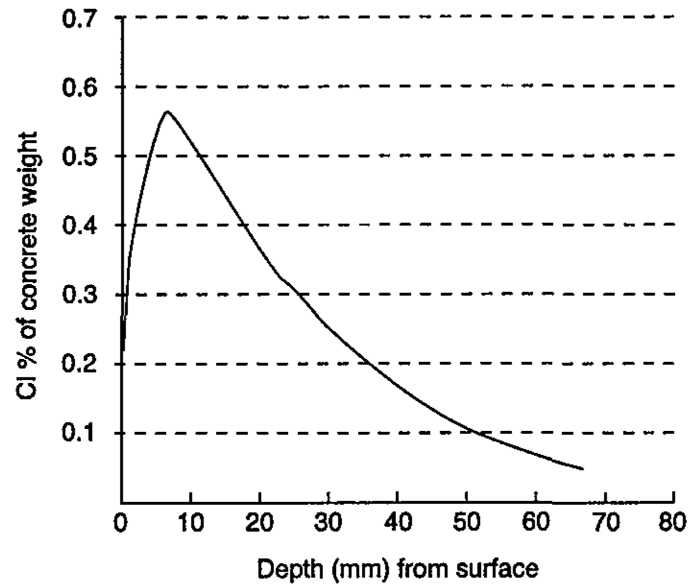

Fig. 18.4. Chloride penetration from concrete surface

for chemical laboratory testing and analysis. The chemical testing should generally include, chloride ion depth profiling and testing, cement content and type, alkali silica reaction testing, sulphate content and carbonation depth testing. Figure 18.4 shows a general profile of the chloride penetration, which has penetrated the hardened concrete, as a percentage of the concrete weight for different depths of the concrete from the surface.

For determination of carbonation depth in concrete, a spray-on indicator with phenolphthalein is used. For indication of the chloride levels, the test procedure is more complicated. The concrete chloride samples are treated with acid to dissolve the cement and the chloride content is determined by titration against silver nitrate. Chloride meters for rapid field tests are available, e.g. Quantab and Hach methods.

Chloride, in harmful amounts, can penetrate further into high-quality concrete than the practical limit of the concrete cover thickness.

Table 18.2. Risk of corrosion due to Cl ion

Cl ion in % of cement weight	Cl ion in % of concrete weight	Risk of corrosion
<0.4	<0.07	Negligible
0.4–1.0	0.07–0.17	Moderate
1.0–2.0	0.17–0.33	High
>2.0	>0.33	Very high

Research has shown that even if the cement content is increased to 500 kg/m^3, the penetration of chloride cannot be prevented. Increased thickness of the cover and/or increased cement content would only delay the penetration of chloride. General experience indicates that a concrete mixture with a w/c ratio of 0.40 or less may give a high resistance against chloride penetration, e.g. low chloride diffusivity. It should be noted, however, that a reduction of w/c from 0.45–0.35 for a concrete based on a pure Portland cement might give only reduced chloride diffusivity by a factor of 2, while a replacement of the Portland cement with a blast furnace slag cement may reduce the chloride diffusivity by a factor of 50. The utilization of pozzolanic materials, such as silica fume, will also improve the chloride resistance. It should be noted, however, that a utilization of blast furnace slag cements or pozzolanic materials will generally make the concrete more sensitive to good curing conditions and, hence, the execution of concrete construction also becomes more important. As a practical guideline, the minimum cement content shall at least be 350–370 kg/m^3. The water–cement ratio shall not exceed a value between 0.40 and 0.45.

Altogether, the best method of ensuring that the natural alkaline protective mechanism is maintained is by providing or mixing a concrete that has the lowest possible permeability. The **concrete cover** to the reinforcement in maritime structures should not, for the different zones, be less than:

Zone 4: above the berth slab = 50 mm
Zone 3: the splash zone = 70 mm
Zone 2: the tidal zone = 100 mm
Zone 1: the submerged zone = 100 mm

Research indicates that the cover thickness needed to prevent a reduction in the passivation due to chloride penetration should be more than twice the cover thickness needed to prevent carbonation.

To estimate the thickness of the concrete cover to the reinforcement, an electromagnetic cover meter is a helpful instrument. The depth to the reinforcement can sometimes be difficult to estimate because reinforcement with small diameter near the surface can give the same depth readings as reinforcement with larger diameter with larger concrete cover. The cover meter readings can estimate the depth to the reinforcement with an accuracy of ±5 mm. Before reading the cover meter should always first be calibrated on the site by testing the meter to locate a reinforcement bar and then doing a control drilling to control its depth.

18.8 Surface coating

The prevention against the chloride ions penetrating into the concrete can also be achieved by using an impermeable membrane on the concrete surface, that is exposed to chloride. A number of different surface protective systems have, over the last years, been developed for prevention of chloride penetration or for retarding the rate of chloride ingress into the concrete. A protective system should be applied immediately after concreting and before any exposure to chlorides if possible.

Solid coating materials have shown that there may, after some years, be problems due to de-bonding and peeling off of the coating. Therefore during recent years, an impregnation of the concrete surface in the form of a hydrophobic treatment has been more widely adopted. These products are available in a form of either pure liquid, paste or gel. Experience has indicated that the efficiency of penetration depends not only on the time of action of the hydrophobic agent, but also that the porosity and the moisture conditions of the concrete at the time of application are very important for the obtained depth of penetration.

Most protective coatings require regular maintenance throughout the life of the structure. Since a concrete based on slag cement is more sensitive to curing, a surface protection of this kind of concrete should be even more important. For the concrete platforms in the North Sea that were given a protective coating of the concrete surface during construction, little chloride penetration has been observed after 15–20 years of exposure. Therefore, in spite of little information being available on the long-term efficiency, current experience indicates that a proper application of a hydrophobic treatment may give a valuable retardation of the chloride penetration.

18.9 Service life

For berth structures, the service life design should be carried out, and appropriate programmes for the life-cycle management and for later regular monitoring of the chloride penetration should be established.

An increase in service life might be achieved by prolonging the initiation period or decreasing the rate of propagation once the damage had started to occur. Therefore, it is important to establish a concrete and a cover depth that provides a durability level which will match the required service life. It is furthermore important to make sure that the level of concrete quality based on laboratory testing is reached on the construction site. The most important factors for durability will be chloride diffusivity and concrete cover. The strategy for the durability design would basically involve the following:

(a) To establish an appropriate concrete mixture in combination with sufficient reinforcement covers, also addressing the appropriate curing conditions in order to meet the required quality level. Necessary backup also has to be addressed if deviations occur on the construction site. Adequate concrete mix will most probably include blast furnace slag cements or pozzolans (fly-ash, silica fume, etc.). Backup systems could be coating or backup cathodic protection if this is acceptable.
(b) Reinforcing steel shall conform to an international standard for reinforcing steel, e.g. EN 10080 or similar. The surface of the reinforcement shall be free from loose rust and deleterious substances, which may adversely affect the steel, concrete, or bond between them.
(c) Stainless steel reinforcement in combinations with high quality concrete mix based on Portland cements. For concrete structures in a chloride containing environment, experience has shown that a partial replacement of the traditional reinforcing steel in slabs and beams with stainless steel can be interesting from a technical and economical point of view. For such applications, steel of type 1.4436 (EU 10088-3) or 316 (ASTM) has been shown to be the most effective, but a simpler type of steel such as 1.4301 or 304 has also shown to be beneficial.

For a chloride-induced corrosion, repairs are both technically more difficult and disproportionately more expensive compared to a regular monitoring of the chloride penetration in combination with a protective coating and/or cathodic protection at a proper stage. Therefore, the following should be emphasized to give an appropriate design for durability of a new berth structure:

(a) Cathodic protection: control of reinforcement corrosion based on cathodic protection and prevention.
(b) Blast furnace slag cements: those countries having extensive experience claim that a high-performance concrete based on a blast furnace slag type of cement will give a much higher durability than that of a Portland cement type of concrete.
(c) Prefabricated concrete elements: for the construction of a berth structure in a rough marine environment, reinforced non-prestressed

concrete elements could be economical both due to construction time and design life. The prefabricated elements constructed under protected and controlled conditions will reduce or avoid problems due to early exposure from splashing and spraying of seawater. Appropriate protection systems for the concrete surface can be applied under controlled and optimum conditions.

18.10 Overloading of the berth structure

The application of too heavy live loads on the berth structure and/or heavy impacts from ships, have caused cracks and damage to the concrete which in turn leads to corrosion of the reinforcement and, finally, breakdown of the structure if not repaired in time.

18.11 In situ quality control

Generally during concrete construction, variations in construction quality such as concrete production, curing conditions and workmanship may produce variations in concrete quality. As a result, the in situ properties may be different from those specified or produced in the laboratory. For all concrete structures, where durability and long-term performance are of great importance, the documentation of the most important in situ properties is important.

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19

Concrete repair

19.1 General

The durability and lifetime of concrete structures are becoming increasingly important. In all repair work the remaining service-life evaluation of the berth structure should be a part of the repair design evaluation.

Damage has been discovered on many old and new structures in the last decades and, unfortunately, on a number of relatively new berth structures. The problem has manifested itself for berth structures particularly in the form of chloride-induced reinforcement corrosion. Contributing factors to these unfortunate experiences were that designers had a relaxed view of the problem and a misguided belief that concrete structures could last forever. Poor concrete quality, insufficient concrete cover and casting errors seem to be the most important reasons for the many cases of damage and the need for repair.

Deterioration of structural elements in the tidal zone and above water is nearly always caused by chloride-induced reinforcement corrosion or freeze–thaw bursting. Deterioration of this type will inevitability occur to some extent on old structures, and is difficult and expensive to repair.

Generally, the repair work should be selected according to the condition of the concrete and the reinforcement, and from the remaining design life and from an economical point of view. When confronted with a deteriorated concrete structure, one often tends to choose a method of repair that involves the lowest cost at the moment, without considering the lifetime of the structure. Unfortunately, this will usually imply repeated repairs and is an altogether wasteful solution.

Underwater, however, damages often occur during the construction period, and reinforcement corrosion is seldom a serious problem provided that the cover is adequate and the concrete has not been washed-out during casting. This means that one must investigate the structure underwater before it is taken over by the owner.

19.2 Assessment

The most important step in the design of repair and or strengthening work is a careful assessment of the existing structure. The purpose of this assessment is to identify all defects and damages, to diagnose their cause and hence to assess the present and future likely adequacy of the structure. The information obtained from structural assessment can then be used to determine whether or not corrective work is required or is economical (when compared to the cost of demolition and replacement) and, if so, how it can best be accomplished. Without prior planning and proper assessment, any programme of corrective work is likely to prove ineffective.

Owing to the safety consequences that have to be considered, an engineer with a broad knowledge and experience within materials technology, deterioration mechanisms, structural behaviour, repair techniques and construction procedures should perform the assessment. The engineer's competence is a vital part of the success of the process.

The assessment could be carried out in several stages. In most cases it is useful to first carry out a general in situ survey allowing an estimation of safety hazards and to give an indication of whether immediate safety precautions are needed. This first-stage survey may help to plan the next stage of the survey, by choosing required type, number and location of future investigation and measurements to be carried out.

The final report, based on the assessment, the laboratory tests, the information from the owner and the structural analyses should contain information regarding the following topics:

(a) structural design data
(b) environmental conditions
(c) information on future use: expected service lifetime of the repair, and required load-carrying capacity
(d) data from visual inspection
  (i) state of corrosion
  (ii) amount of spalling, cracking and patches
(e) data from in situ and laboratory investigation
  (i) concrete strength
  (ii) concrete cover

   (iii) chloride profiles
   (iv) half-cell potential readings
- (f) load-carrying capacity of the structure
- (g) description of each structural element and the cause of deterioration
- (h) evaluation of different repair techniques
- (i) need for strengthening
- (j) economical evaluation of different repair strategies
- (k) conclusion.

19.3 Repairs of concrete

The choice of method to be used for repair of deteriorated concrete will, in each case, depend on the zone in which the deterioration is found and the cause of the deterioration. It is therefore essential to establish what has gone wrong before implementing a repair procedure. By careful evaluation of the extent and the cause of the concrete deterioration, the procedures for the repair work will also accomplish one or more of the following:

- (a) restore the structural strength
- (b) increase the structural strength
- (c) improve the concrete surface appearance.

In connection with repairs on old concrete, the electrolytic conditions in the structure should be altered as little as possible. Much research has been done to find ways to seal the concrete surface against salt water (chlorides) penetrating into fresh concrete and reaching the reinforcement. Means of protection, such as bitumen, epoxy, etc., applied on beams and slabs have not proved to last for more than 5–10 years. The reason is probably that the coating is too tight, leading to condensation inside and subsequent freezing and scaling off. To obtain a satisfactory coating, it must be sprayed on a clean, not too smooth surface so that it has a good overall adhesion. However, the coating itself must not be so impermeable that condensation water cannot escape, i.e. the coating must be able to 'breath'.

Galvanized reinforcement has also been tried against corrosion, but the galvanization is expensive and galvanized bars are not produced in long lengths. The bond between galvanized steel and concrete is more or less the same as for non-galvanized steel. Therefore cathodic protection of the reinforcement should be the alternative but, in practice, care should be taken to keep the potential value constant all through the reinforcement.

Table 19.1. Methods of concrete repair

Methods of repair	Zone 1	Zone 2	Zone 3
Tremie pipe concreting	×	×	
AWO concrete	×	×	
Injection	×	×	×
Micro-concrete	×	×	×
Special epoxy	×	×	×
Shotcrete		×	×
Catodic protection embedded in shotcrete			×

In the tidal zone (Zone 2) the formwork should normally not be removed but remain as an additional protection. Research has clearly shown that such 'permanent' formwork is an efficient measure against freeze–thaw attacks. Concrete that is permanently submerged (Zone 1) needs no such special protection and all formwork should be removed so that the concrete surface can be inspected.

Therefore routine control should always be established so that faults can be detected and repaired as soon as possible. In the long run the most economic 'protections' are correctly composed concrete and the cast concrete having a correct cover thickness outside the reinforcing bars.

Generally speaking, the methods of repair indicated in Table 19.1 are especially suitable in the various zones.

Regardless of zone, all poor concrete must be cut or chiselled off and the concrete surfaces cleaned. Chloride-infected concrete approximately 2 cm behind the main reinforcement in Zones 2 and 3, must be removed before further repair. Rust must be carefully removed, preferably by sandblasting, before fresh concrete or epoxy is applied. It is quite possible to carry out compressed air chiselling, sandblasting and pressure water jet washing underwater to about 20 m depth.

In order to avoid growth of algae, etc. on the cleaned concrete surfaces below water and thereby reduction of adherence between old and fresh concrete, the placing of fresh concrete must be carried out immediately after the cleaning, as shown in Fig. 19.1.

19.4 Repairs in Zone 1

No matter whether the deterioration has taken place on old or fresh concrete, the choice of repair method in Zone 1 will be the same. Repairs in this zone are made by tremie pipe concreting, injection,

Fig. 19.1. Cleaned concrete surface before repair

micro-concrete or special epoxy. As these methods involve diving, the work must be planned in such a way that it can be carried out in a simple and straightforward manner underwater.

19.4.1 Tremie pipe concreting

Berth columns and structural elements of similar dimensions being repaired by the tremie pipe method ought to be provided with a mantle all around in order to give the newer part of the element a good bond with the old. This applies even if deterioration is found only on one side. Additional reinforcement must be provided or, if this is not needed for structural strength reasons, reinforcement net should be applied to keep the concrete in place.

When deterioration has taken place at the joint between concrete and bedrock, the latter must be cleaned of old concrete, and it should be carefully considered whether additional bolts in the rock ought to be provided together with the concrete mantle. The formwork for the mantle should be carefully tailored to the rock surface in advance, so that the diver does not have to pack between formwork and rock during the concrete pouring.

The thickness of the mantle must be sufficient to provide room for the tremie pipe between the additional reinforcement and the formwork.

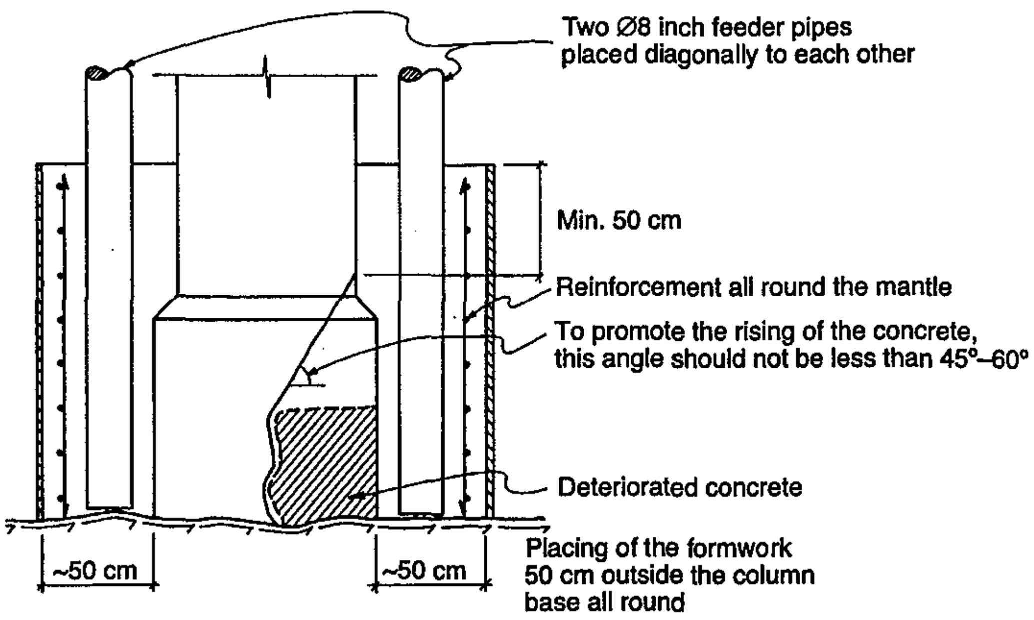

Fig. 19.2. Repair of column base on rock — tremie pipe method

For the concreting of such a mantle, two tubes should be used, placed on opposite sides of the column. Figure 19.2 illustrates the repair of a column base on rock.

If the part to be repaired is not very far below the berth slab, flexible plastic pipes through which the concrete is pumped to the place of repair and placed in the same manner as for tremie pipe concrete, can replace the tremie pipe of steel and holes can be made for them in the slab.

If the AWO concreting technique is used the procedure will be the same.

19.4.2 Injection

Special contractors who have the necessary equipment for injection concreting generally perform this method. The method is normally used for older structures undergoing comprehensive repair and maintenance works. One reason for this is that in columns recently concreted, deteriorations are usually discovered before the slab is concreted, so that there is access for tremie pipe equipment from above. However, in new heavily reinforced structures the injection method can be a good alternative.

When using injected concrete the thickness of the concrete mantle can be reduced as compared to the thickness required in tremie pipe concreting. After placing the formwork outside the reinforcement, the

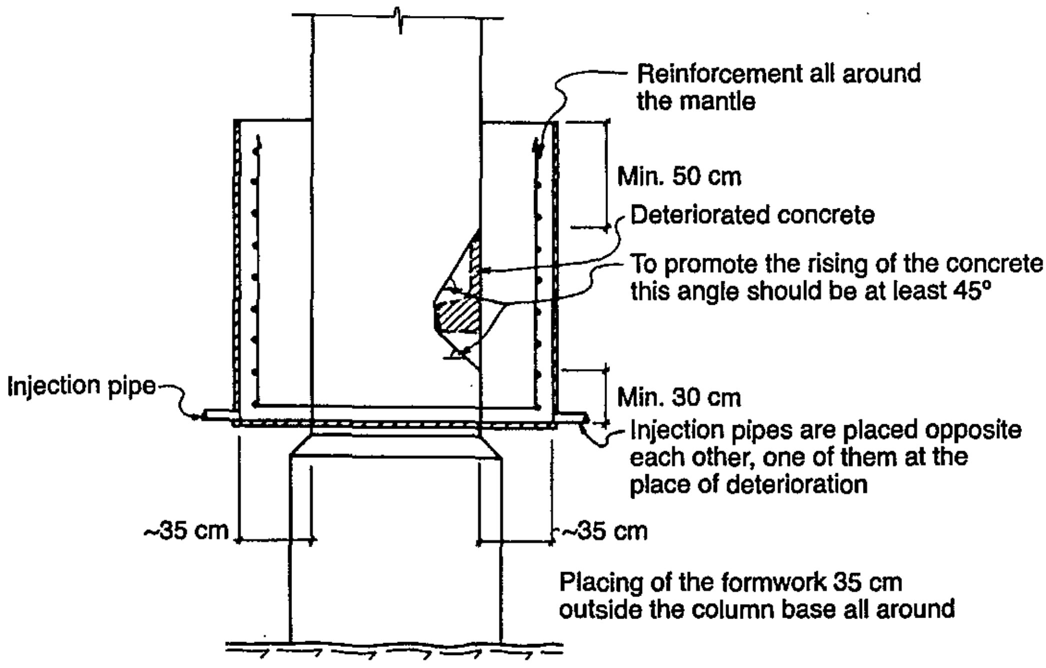

Fig. 19.3. Repair just above column base, injection concrete method

injection pipe is put in place, and finally the aggregates. Figure 19.3 illustrates a repair just above a column base.

The coarse aggregates should consist of cleaned and sieved gravel and crushed stone with a minimum size, that is 8–10 times the maximum grain size of the injection mortar. The specified minimum size of coarse aggregates is 2 cm and the maximum size of aggregates should be 5 cm. The injection mortar should have a cement–sand ratio of 1 : 1 by weight plus expanding and stabilizing agents.

19.4.3 Micro-concrete

The micro-concrete method represents a simplification of the injection method. The procedure is the same apart from the formwork not being filled with aggregates, only with injection mortar. The micro-concrete method lends itself to smaller repairs in particular. Figure 19.4 shows the repair of a wall with micro-concrete.

If the part to be repaired is so long that two injection pipes above each other are needed, the procedure is the following: first, the mortar is injected by way of the lowest pipe until the concrete level reaches the next injection pipe.

The lower pipe is then closed and the injection hose is disconnected. While the hose is still full of mortar, it is moved up to the following injection pipe and the process is continued. When starting the injection,

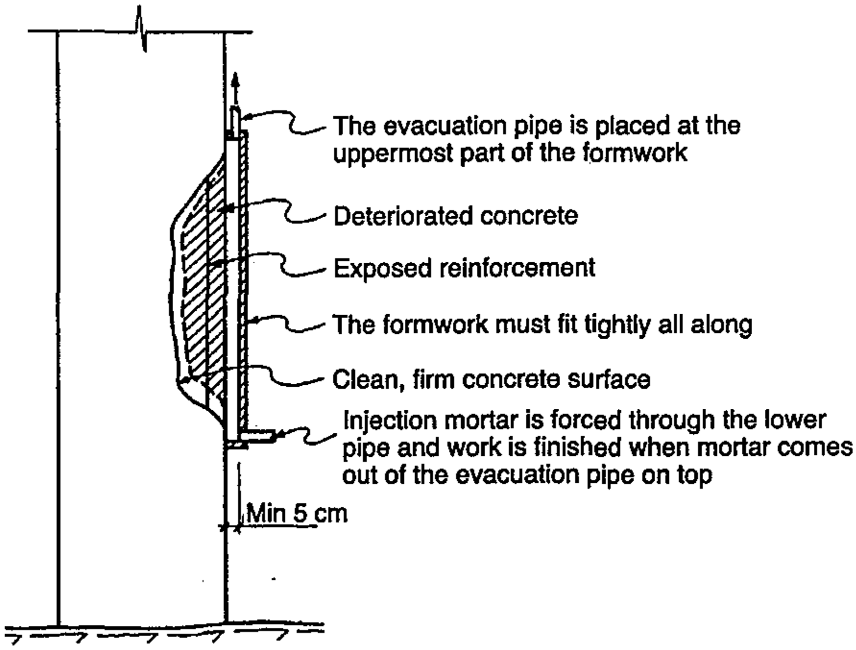

Fig. 19.4. Repair of a wall with micro-concrete

the pressure must be adjusted with a view to avoiding a too-fast flow of mortar and thereby a washing-out of its finer components.

19.4.4 Special epoxy

For smaller repairs of volume up to say 0.01 m^3, the use of epoxy may prove successful provided the curing temperature is above 5 °C. If the volume to be repaired is approximately 0.25 l, the diver will be able to carry out the repair with his hands (using discardable gloves) and using a plastic epoxy smooth material. The pot life of this material is about 20 min at 20 °C depending on the type of epoxy.

19.4.5 Rescon method

Cement and epoxy resin products have been used in underwater repair, but used alone both have drawbacks. For example, in contact with water cement can be leached out from the repair grout, leaving a low-strength product. Also, epoxy resin mortars are limited by high cost and their mechanical properties, which differ substantially from those of concrete. For example, the Norwegian Rescon Method combines both repair materials and utilizes their best properties. For example:

(a) The E-modulus and thermal coefficient of expansion of the cement grout is similar to that of concrete.

(b) The repair is more economic in material costs when compared to an equivalent volume of epoxy mortar.
(c) Cement wash-out is eliminated by the epoxy resin in the Rescon method.
(d) The epoxy resin ensures good adhesion between the repair grout and the parent concrete (up to 2.5 MPa).

Procedure

(a) Remove the damaged or eroded area, leaving only sound material and prepare a suitable bonding surface using grit blasting or water jetting.
(b) Construct shuttering using smooth, preferably transparent, sheets. Make tight at the base and side, using foam strips, mechanical fixing and underwater putty. The shuttering is positioned slanting away from the structure at the top to allow access for the hose down which the repair materials are pumped from the surface.
(c) The epoxy resin is mixed at the surface and pumped into the base of the mould to an approximate depth of 10–20 cm down the hose, which reaches into the lowest part of the mould.
(d) The epoxy is immediately followed by an expanding cement grout, which displaces the epoxy resin from the base of the mould. This action coats the structure with epoxy improving the adhesion, coating the shuttering and ultimately giving a protective epoxy coating to the grout while maintaining a layer of epoxy on the surface of the rising grout, preventing cement leaching.
(e) When the epoxy and grout have finally cured (the increase in temperature of the curing cement also helps to cure the epoxy), strip the shuttering for reuse.

19.5 Repairs in Zone 2

The above descriptions of repairs in Zone 1 are valid also for repairs in Zone 2. In addition, the shotcrete method can be used in Zone 2.

If the repair has to be carried out underwater, the procedure will be as for Zone 1. However, one should aim at repairing under dry conditions. This can be achieved by installing a waterproof box or 'cofferdam' serving as a working platform around the column, or alongside the wall to be repaired. One can thus obtain a dry working place for chiselling off the deteriorated concrete and repairing it with shotcrete. The methods are illustrated in Fig. 19.5. For repair of a column, two steel box halves are

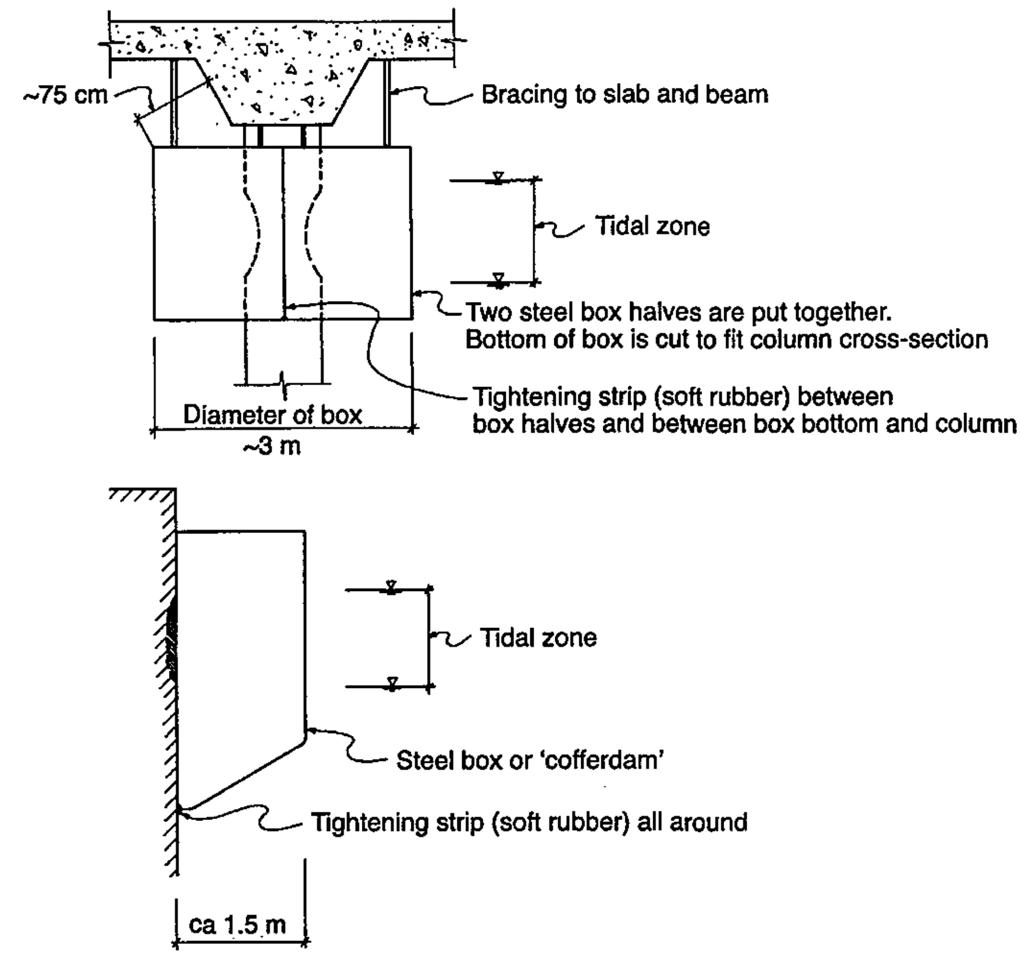

Fig. 19.5. Waterproof box or 'cofferdam'

lowered down on each side of the column and then put together and fastened underneath the berth slab. The box must be high enough to cover an ample space below the damaged area and also above the tidal zone. Soft rubber is used as sealing material between the two parts of the box and between the box and the column. The box is then pumped empty.

If more than one column is to be repaired, it pays to put some effort into the design of the box system, with a view to making it easy to install and remove the boxes for re-application at several columns.

19.5.1 Shotcrete or gunite

The working platform described above must be sufficiently large for the working operations required. Approximate platform dimensions are shown in Fig. 19.5.

When the concrete surface has been carefully chiselled and sand-blasted, the reinforcement, if required, is put in place and shotcrete is applied until the column has regained its cross-sectional dimensions. It is not recommended to cover the old concrete surface with a coating of epoxy glue to obtain a better cohesion, because the epoxy is eroded away by the dry mix shotcrete. Where the deteriorated column has become 'hour-glass' shaped and additional reinforcement has to be applied, concrete should be sprayed on to the old concrete before the new reinforcement is installed.

It is better to spray two layers of 2–3 cm thickness of concrete than to add an accelerating agent to the concrete and spray a 4–5 cm layer in one operation. Due to material loss, however, the latter method is more usual, notwithstanding the fact that accelerating agents tend to reduce the compressive strength of the concrete. The extent to which such reduction takes place depends on type and make of the agent used. A strength reduction of 50 per cent has been recorded in concrete with a 5 per cent agent content. Generally, concrete of high compressive strength also has better adhesion and properties than low compressive strength concrete.

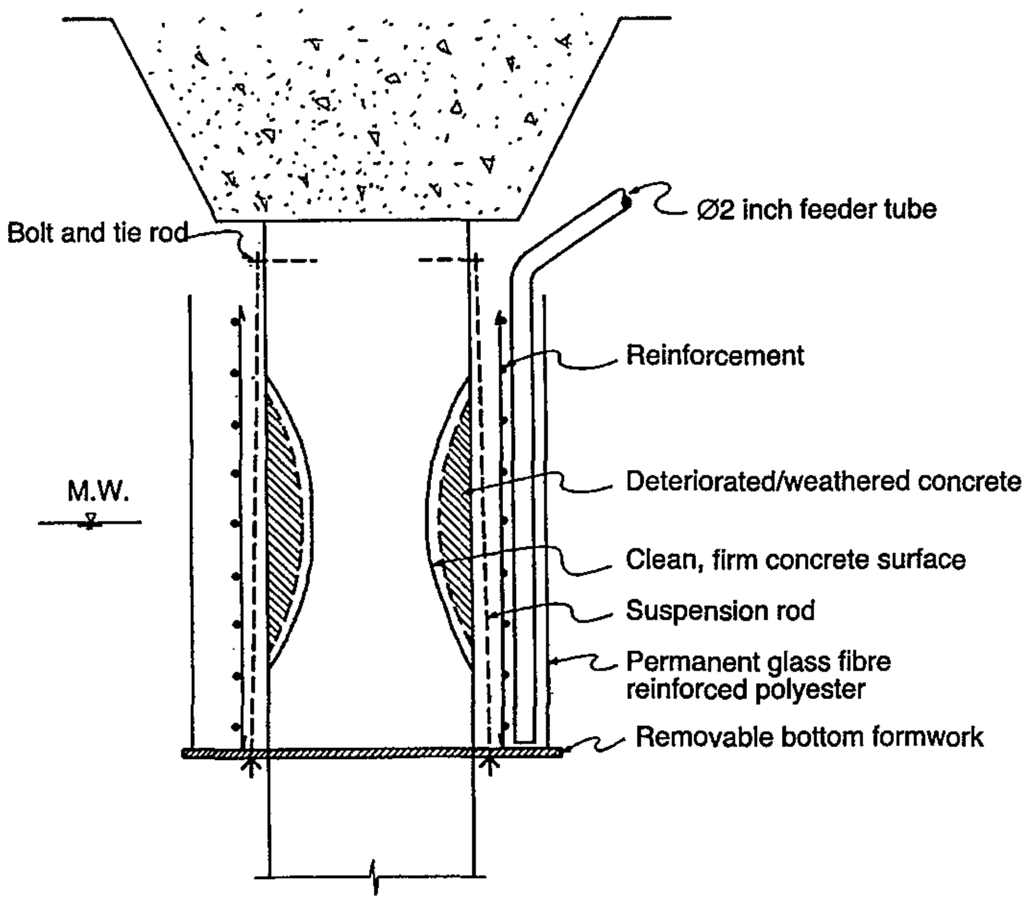

Fig. 19.6. Example of feeder tube

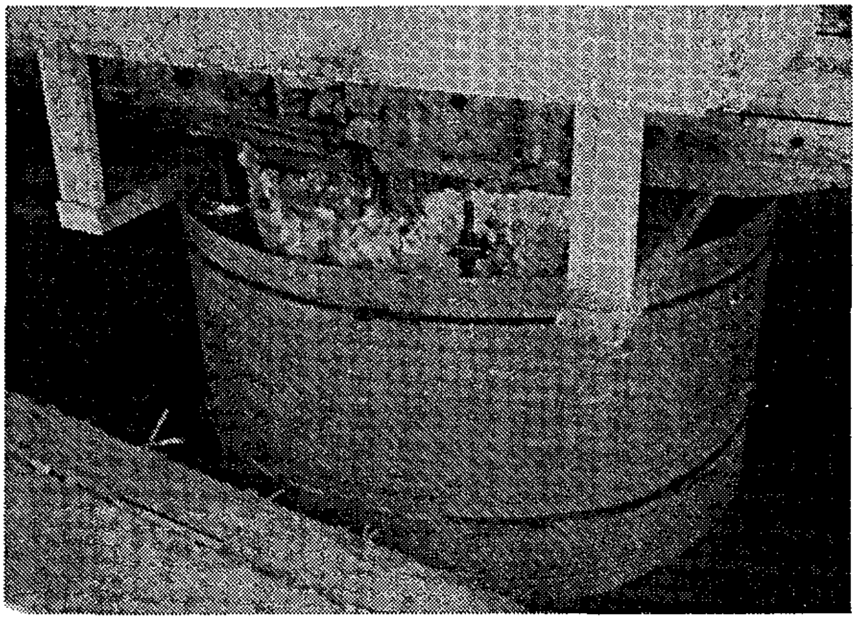

Fig. 19.7. Glass-fibre formwork before being filled with concrete

19.5.2 Tremie pipe concreting

Figure 19.6 shows, in principle, a tremie pipe concreting repair where the formwork hangs on bolts drilled into the column higher up. The formwork bottom consists of two semi-circular formwork plates, with a cut-out for the column, hanging in bars from the bolts above. The sides of the form consist of two semi-cylinders of glass-fibre reinforced polyester. These are later utilized as permanent formwork.

The concrete is known as pea-shingle concrete with a slump of 16–18 cm, which is placed on the bottom of the form through a 2″ diameter pipe from a concrete feeder screw. During the concreting operation, the pipe is not moved before the concrete overflows the top of the form. Knocking on the sides of the form with a hammer or similar is done in order to compact the concrete in the formwork. The amount of new reinforcement to be used depends on how much steel has rusted away, but a steel mesh should be applied in any case.

Figures 19.7 and 19.8 show the glass-fibre formwork before and after being filled with concrete.

19.6 Repairs in Zone 3

In order to successfully repair a berth structure, particularly in Zone 3, it is important to have a full understanding of the deterioration process that is occurring in the structure. This requires full investigation of the concrete with mapping of the condition to gain a full understanding

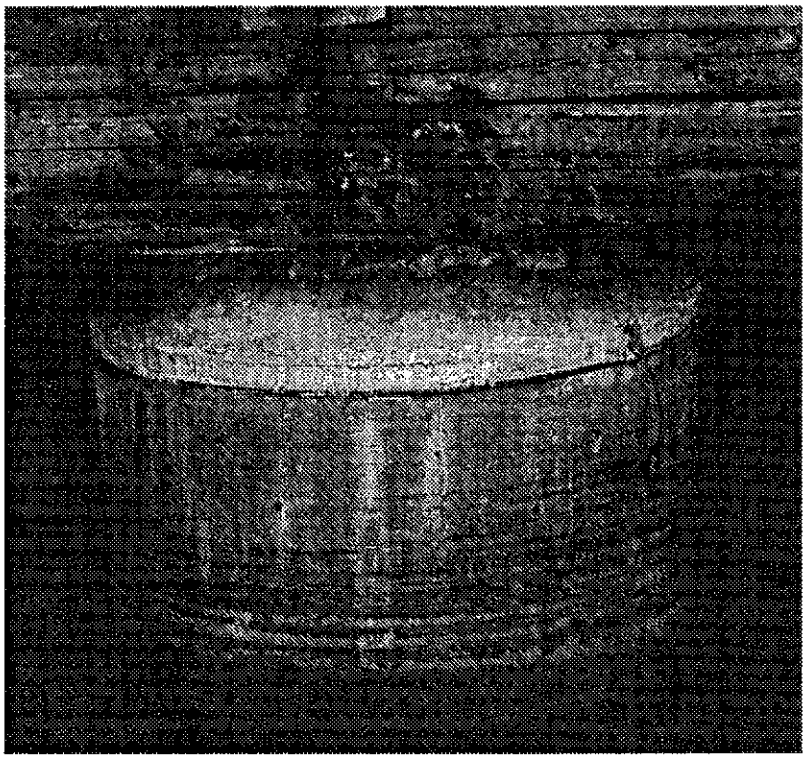

Fig. 19.8. After the repair of column

before one is able to select the most appropriate technical repair method.

The understanding of the deterioration mechanisms and the structural behaviour is a prerequisite for developing rational and sound repair strategies. The only way of influencing a deterioration process is by influencing the parameters governing this mechanism.

In the evaluation to decide the repair type for the structure, one should always consider the following strategies.

(a) Postpone the repair, and monitor the structure.
(b) Recalculate the structure, and reduce the load-bearing capacity.
(c) Repair the structure to increase the service life.
(d) Strengthen the structure.
(e) Rebuild parts or the whole structure.
(f) Demolish.

Since repairs are, in general, very expensive, the strategy or combination of strategies that is both technically and economically favourable should be chosen; also that the structure after the repair will have as low a life-cycle cost as possible.

Marine structures that have deteriorated as a result of chloride-induced reinforcement corrosion can be repaired by one of the following

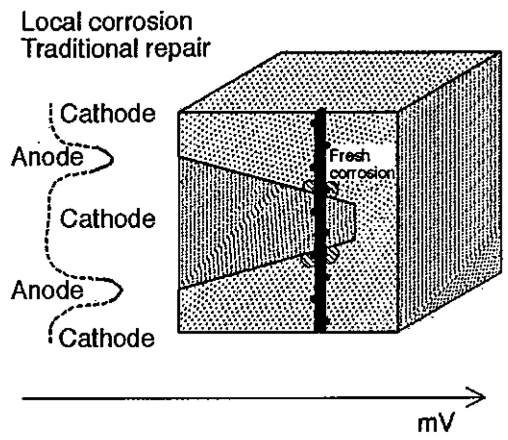

Fig. 19.9. The patch repair method

methods. The commonest and most proven methods, that have been adopted over the last decades, are:

(a) patch repairs
(b) cathodic protection
(c) chloride extraction.

Experience from repair work shows that the commonest mistake is to wait too long before the repair work is carried out. This is a very expensive way to operate the structure and a lot of money could be saved by providing a correct assessment and optimum start of the repair. This fact is mainly explained by the high cost of removing and replacing damaged concrete.

19.6.1 Patch repair

The patch repair method has traditionally been used over many years, but does not represent a successful long-term repair method. In many cases it has been accepted that the repaired berth structure will eventually need a new repair since the method is neither technically nor economically viable. This often leads in the end to new severe deterioration and later, perhaps, demolition and replacement of the structure.

The traditional way of repairing spalled and cracked concrete in Zone 3, due to chloride induced corrosion, is using the patch repair procedure as shown, in principle, in Fig. 19.9. For instance, in a concrete beam in Zone 3 that has deteriorated due to corrosion of the reinforcing steel, one cannot simply replace the poor or deteriorated concrete with fresh concrete. The fresh concrete is not chloride-ionized and will probably cause an accelerated corrosion when coming into contact

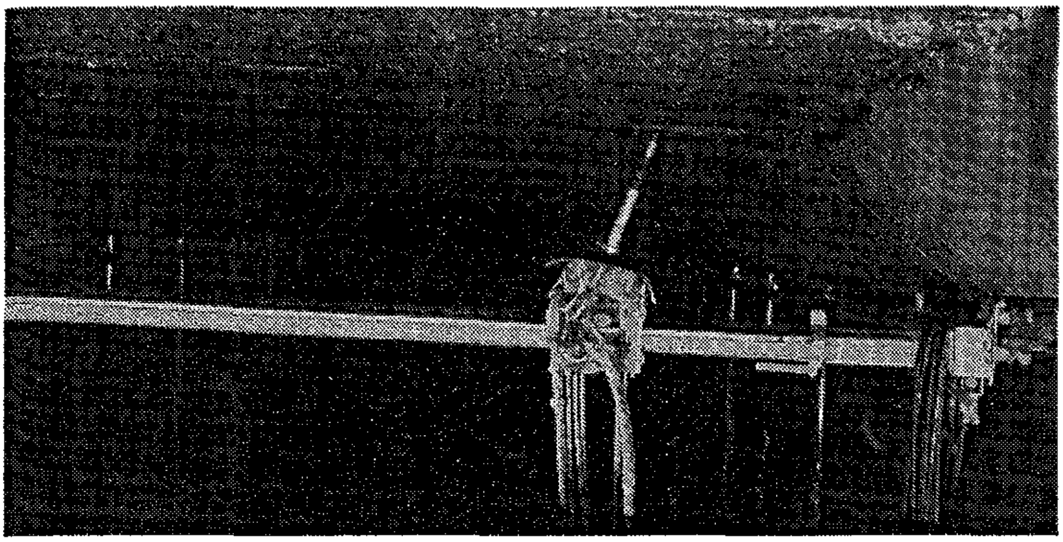

Fig. 19.10. Waterjet for removing the loose concrete. Photo courtesy of Betongfornyelse A/S, Norway

with older chloride-ionized concrete. Normally only the damaged (anodic) area is repaired, and chlorides in the neighbouring zone may after a short time move the anode to these zones, and corrosion continues. Therefore the patch repair method should consequently be applied to new structures only, where chlorides have not generally reached the level of reinforcement.

Such repair will not last long and has not taken into account available knowledge of the chloride corrosion mechanism.

Generally the patch repair should be carried out in a way that all concrete containing chlorides above a certain content is removed around and to 20–30 mm behind the reinforcement. This is in fact perhaps the most expensive way of repairing and it is important to be aware that new damage may occur within 5–10 years.

The patch repair method may be chosen when one has to strengthen the structure temporarily for safety reasons, or where the remaining service life for the structure is short. Either way the repair should be carried out by proper removal of damaged concrete, cleaning of the reinforcement and by using a repair cement mortar of proven good quality.

Deterioration of old concrete in Zone 3 is usually caused by corrosion of the reinforcement. Longitudinal cracks and wounds in connection with corroded steel bars can often be seen on the bottom parts of older rectangular high beams. Figure 19.10 shows a waterjet arrangement for removing all loose concrete around the reinforcement.

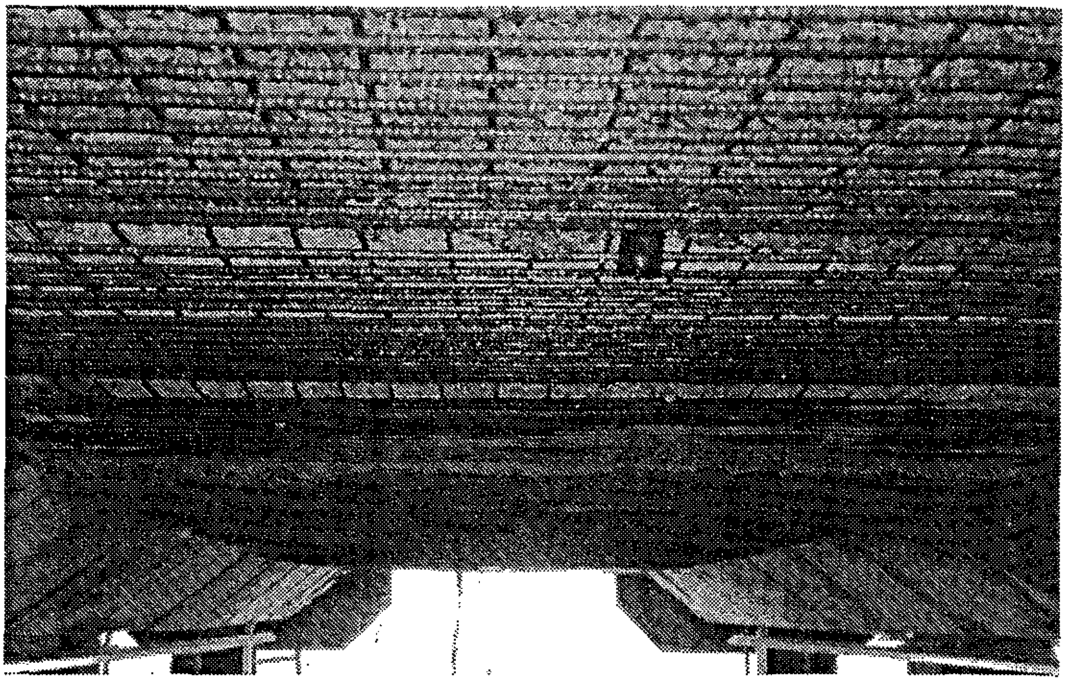

Fig. 19.11. All loose concrete has been removed from the slab. Photo courtesy of Betongfornyelse A/S, Norway

When all loose concrete and chloride-infected concrete to approximately 2 cm behind the main reinforcement, as shown in Figs 19.11 and 19.12, has been removed and the concrete and the reinforcement have been sandblasted, the decision must be taken whether additional reinforcement should be installed before the existing reinforcement is covered by shotcrete.

The injection method is used only for the repair of beams. Figure 19.13 is a typical example of such repair. The sloping sides of the beam will ensure a better and easer filling of the form. The whole length of the beam ought to have this new cross-section, not the deteriorated parts only. This applies also when the shotcrete method is used. Stone aggregates and injection mortar are as described for repairs in Zone 1. Also micro-concrete can be used as described for Zone 1.

If the cross-sectional area of reinforcement has been reduced by corrosion to such an extent that additional reinforcement has to be installed, or if it has been necessary to remove deteriorated concrete behind existing reinforcement, the spraying of concrete behind the bars can prove difficult. In order to avoid cavities in this concrete, the bar diameter ought not be more than 12 mm. However, in beams 25 mm diameter bars can hardly be avoided and therefore micro-concrete should be used.

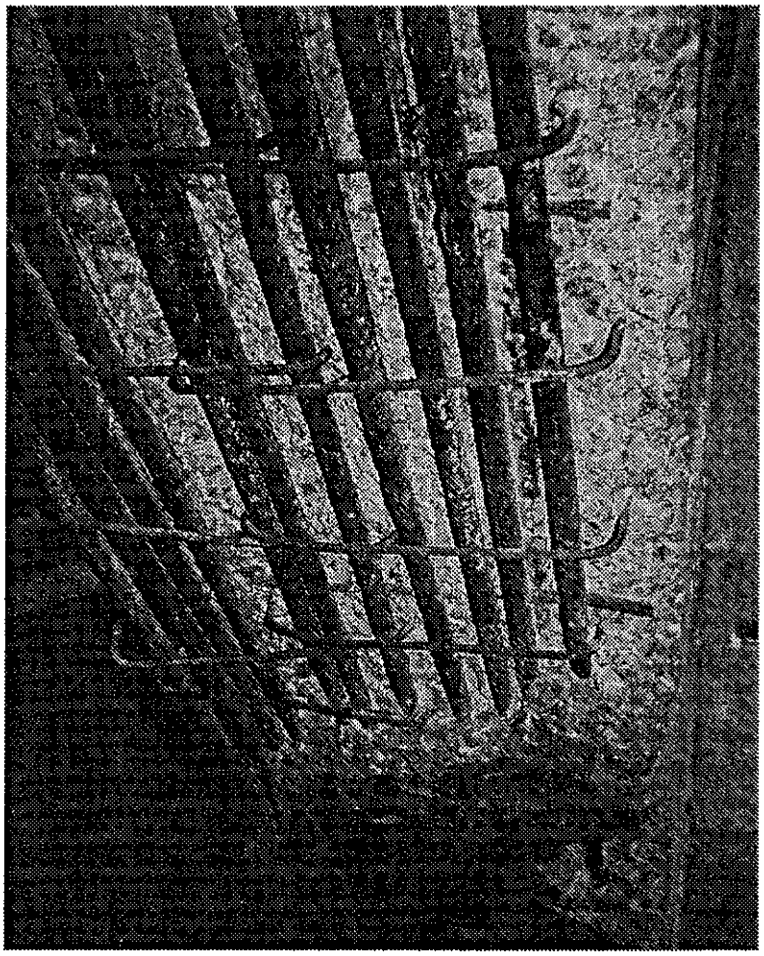

Fig. 19.12. All loose concrete has been removed from a beam

19.6.2 Cathodic protection

Although the principle of cathodic protection (CP) has been around since the 1820s, its use in repair of reinforced concrete did not start until the late 1950s.

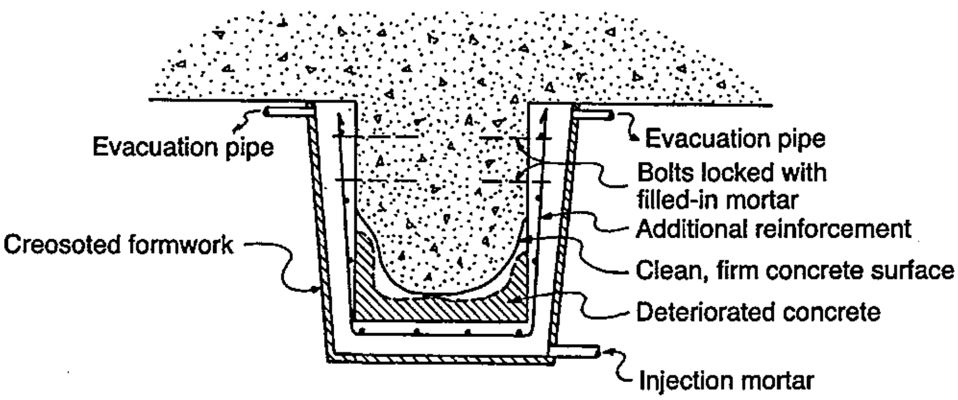

Fig. 19.13. Repair of beam

Chloride corrosion is a very serious type of deterioration and, since a patch repair is not expected to last long, the most reliable repair method nowadays is to use CP. This repair method is the only way to stop the corrosion rate, and has by experience over the last decades proven to be a very effective repair method.

In principle there are the following two types of cathodic protection systems available on the market nowadays:

(a) The **sacrificial anode system**, which is generally used only below mean water level, and is almost maintenance free. For example, zinc alloy sacrificial anode bracelets fixed to the berth structure at approximately LAT.
(b) The **impressed current system** using various types of anodes, as shown in Table 19.2, is usually used above mean water level, and requires regular monitoring and a degree of expertise to operate it with maximum efficiency.

Figure 19.14 shows a principal sketch of a cathodic protection system.

Impressed current cathodic protection works by passing a small direct current (DC) from a permanent anode fixed on top of the surface or into the concrete, to the reinforcement. The power supply passes sufficient

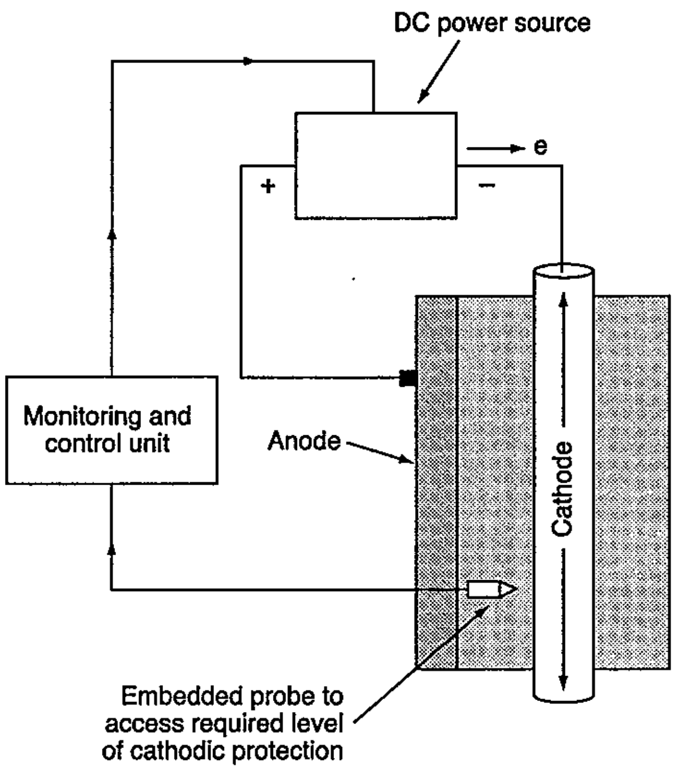

Fig. 19.14. The principal sketch of a cathodic protection system

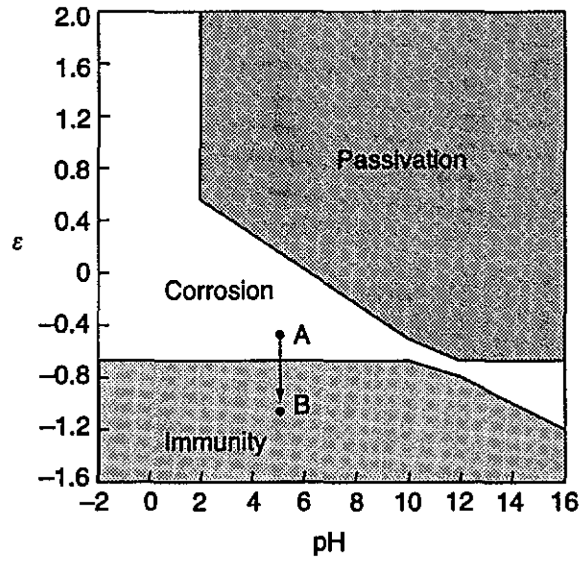

Fig. 19.15. The Pourbaix diagram

current from the anode to the reinforcing steel to force the anode reaction to stop and make a cathodic reaction the only one to occur on the steel surface.

The Pourbaix diagram explains the cathodic protection system. By applying a small impressed current, the potential will move, for example, from A to B in the negative direction, as shown in Fig. 19.15. The steel will then become immune and corrosion will stop due to the cathodic protection system.

There are different methods of installing cathodic protection, as shown in Table 19.2. The selection of the most appropriate type of

Table 19.2. Methods of installing cathodic protection

Anode material	Expected service life	Current density (mA/m^2)	Comment
Titanium mesh	+25 years	10–20	Durable established system. Main problem may be overlay application and increased weight
Titanium ribbon	+25 years	10–20	Main problem may be overlay application
Conductive mortars	+25 years	20–50	The system will secure an even distribution of current, but it is important to avoid short-circuiting
Discrete anodes (embedded probes)	+20 years	10–20	Durability of the backfill may be a problem, but relatively easy to maintain

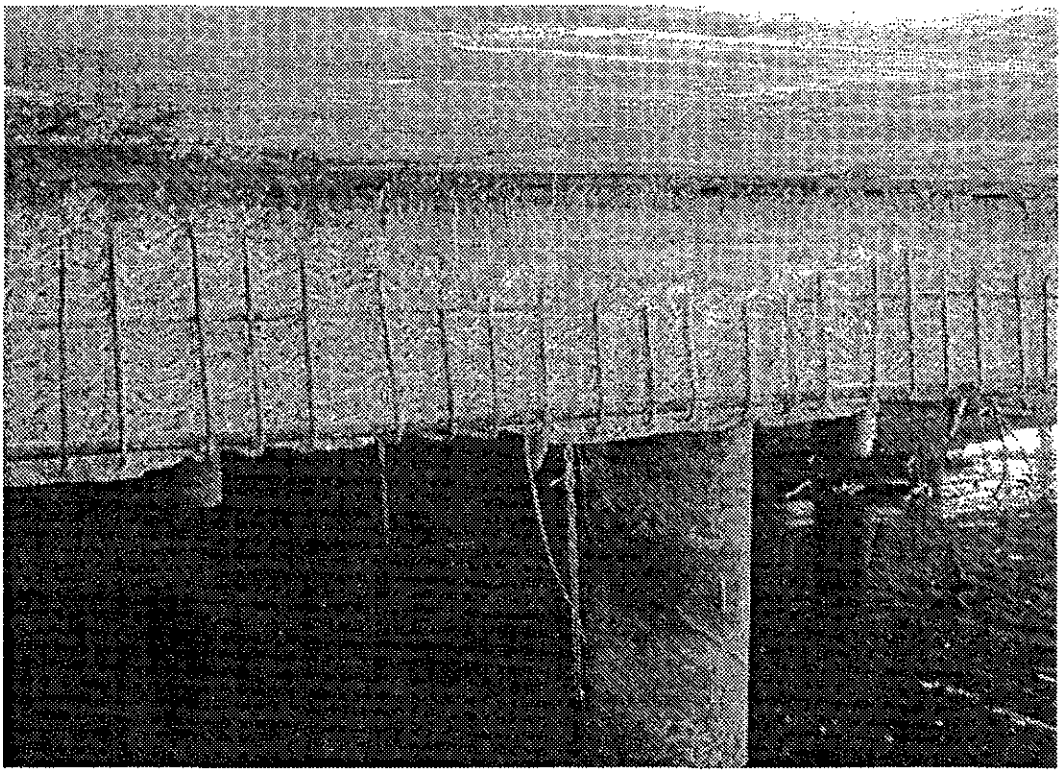

Fig. 19.16. Removing delaminated concrete and rust from the beam

cathodic protection system will depend on the nature of the structure, economy, anticipated service life for the berth structure, environmental conditions and the maintenance capability of the port owner. There are a lot of different types of anodes available on the market and new ones are constantly being developed. For marine structures one should choose robust systems and the following anode systems are recommended.

To reduce repair cost it is recommended to carry out the repair when the amount of corrosion and spalling is low. With cathodic protection the requirement for breaking out sound but chloride-contaminated concrete is not necessary. Only delaminated concrete and rust product has to be removed, to ensure a homogeneous concrete material, as shown in Fig. 19.16. The replacement of new concrete should generally closely match the original concrete.

The titanium mesh and ribbon systems are mechanically fixed to old concrete surface, as shown in Figs 19.17 and 19.18. After the titanium mesh or ribbon system has been fixed to the concrete surface, it is covered by a 20–30 mm thick layer of sprayed concrete, as shown in Fig. 19.19.

A conductive mortar system has been developed over the recent years, and could be particularly economic where large areas have to be repaired.

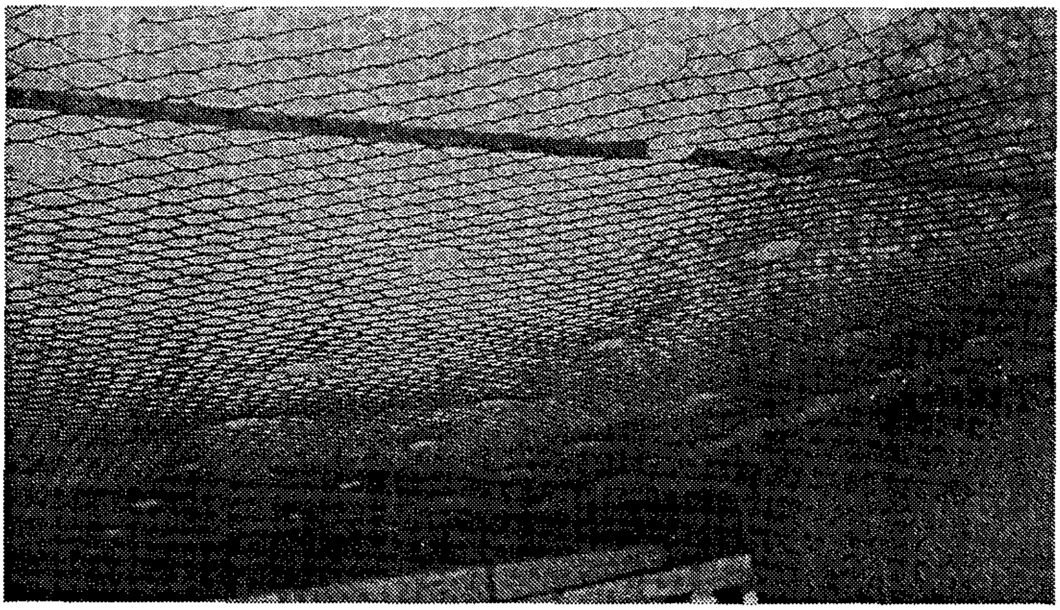

Fig. 19.17. Installation of titanium mesh

The conductive mortar system uses nickel coated carbon fibres to provide conductivity. It is applied by using similar equipment and methods as for sprayed gunite concrete.

If the CP system is designed, installed and maintained according to well-established knowledge, it is the best repair system on the market and the only system that can stop corrosion.

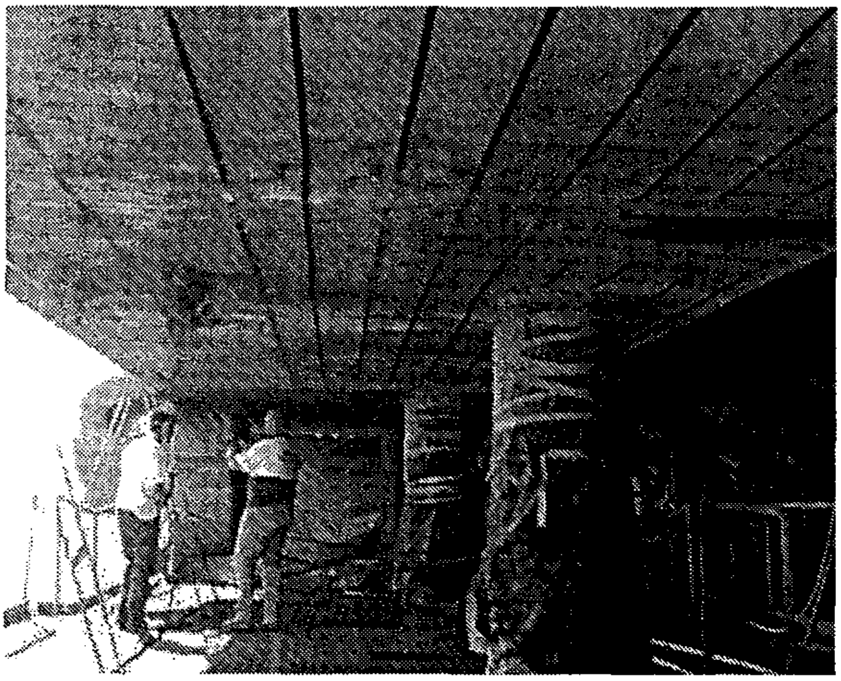

Fig. 19.18. Installation of titanium ribbon. Photo courtesy of Entreprenørservice AS, Norway

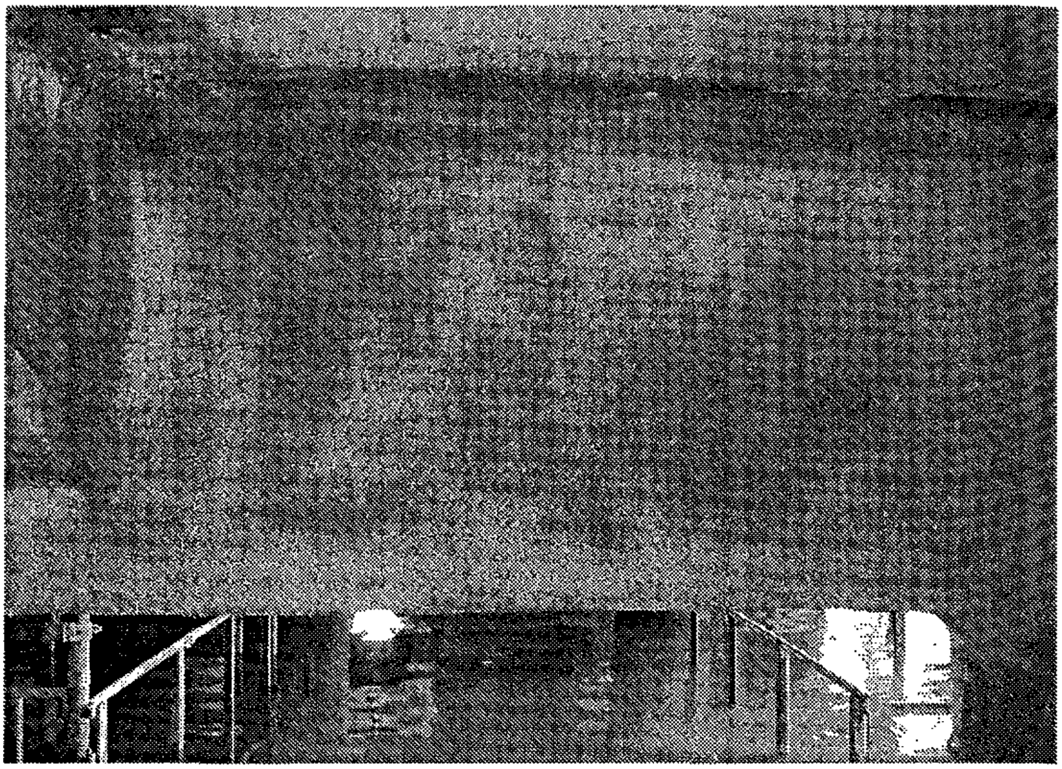

Fig. 19.19. The anode system covered by dry sprayed concrete

Design, preparing of tender documents and installation of such a system requires specialists. Experienced personnel should monitor the system. It is, therefore, recommended that the owners sign a maintenance agreement with the supplier or another experienced consultant company to look after, adjust and run the system for the required life span.

19.7 Chloride extraction

Desalination is a method that may remove some of the chlorides from the concrete by electrochemical means. This method has been reported as successfully used in some projects. Nevertheless, one should bear in mind that the concrete has to be properly protected from chloride ingress after removal. This method may be the right choice for some projects, but the life span of the repair must be expected to be lower than for the well-documented cathodic protection systems.

19.8 Service inspection

For all concrete berth structures and equipment both periodic and regular inspections are necessary so that any damages may be detected in good time so that costly repair and renewal of the structure can be avoided.

The owner should perform a service inspection of the completed berth structure at least every third year. The inspection should both give an evaluation of the berth structural condition and provide a maintenance procedure if necessary. The areas of special interest for inspection will normally be the splash zone, construction joints, previously repaired areas and areas of vital load transfer. The basis for the inspection and observations should be the construction's drawings 'as built' and, if performed, notes from the latest service inspection.

Usually the first inspection of any structure would be at the takeover or acceptance of the structure, and the second inspection should be at the end of the guarantee period. All results of inspections should be recorded for later necessary evaluations of maintenance measures. The service reports from the inspection should also be sent to the designer and the contractor for their information.

19.9 Condition of a structure

To give the owner of a berth structure an idea of the condition of the structure before the repair work starts, the following classification system can be used. The system depends on a thorough inspection of the columns, beams and slabs, and gives each part a mark according to its condition. The mark will also indicate the expected lifetime and live loading, as shown in Table 19.3.

Table 19.3. Expected lifetime and live loadings

Marks	Definition	Remaining lifetime of the structure	Live loading
1	Satisfactory	40–50 years	As designed
4	Damaged but worthwhile to repair	10–20 years	Nearly as designed
7	Costs more to repair than to rebuild	5–10 years	Has to be substantially reduced
10	Damaged and cannot be repaired	0–1 year	No live load

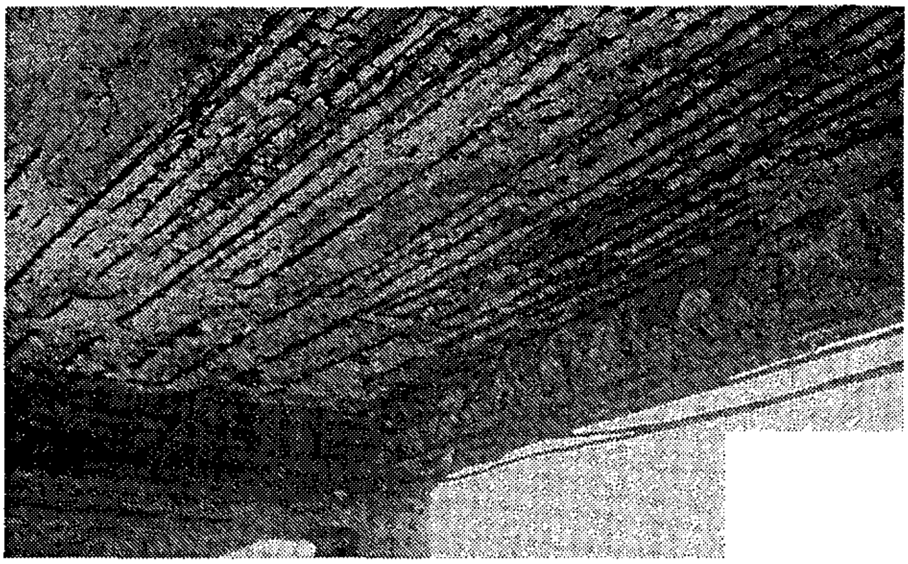

Fig. 19.20. Beyond repair

Based on this system, one can give each of the columns, beams, slabs, etc. an average mark, and the berth itself can be given an average total mark which will give the owner a very rough indication of the condition of the berth structure. Figure 19.20 shows a berth slab and beam beyond repair when no maintenance and repair work have been carried out during the structure's service life.

19.10 Costs of repairs

Owners tend, unfortunately, to choose a method of repair that involves the lowest cost at that moment in time, without considering the expected lifetime of the structure. Irrespective of whether the berth structure to be repaired is new or 40–50 years old, in both cases the cost of repair and the assumed remaining life of the berth structure should be considered as a whole.

To estimate the costs of repair on the basis of visual inspection is difficult because the extent of deterioration first becomes apparent when all deteriorated concrete has been removed. The costs therefore tend to be higher than expected.

Usually, the repair work is paid on account, i.e. the contractor gets all his direct expenses reimbursed, plus a fee. The latter can be a percentage of the direct expenses or a fixed fee, or a combination of these, according to advance agreement between the Client and the contractor. However, the cost and the quality of the work depends first of all on the expertise and management employed by the contractor.

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20
Ship dimensions

20.1 General

The following ship definitions are used in the design of ports and harbours:

Deadweight tonnage (DWT): the carrying capacity of the ship, namely the total weight of cargo, fuels, fresh water, etc.

Gross registered tonnage (GRT): the total internal capacity of the ship divided by 100 ft^3 or 2.83 m^3, depending on the application of relevant laws and regulations.

Displacement tonnage (DT): the total weight or mass of the ship is obtained by multiplying the volume of water displaced by the ship by the density of the water. The displacement tonnage for mixed cargo and bulk-cargo ships is roughly equal to 1.2–1.4 times the deadweight tonnage and equal to 2.0 times the gross registered tonnage. For passenger ships, the displacement tonnage is roughly equal to 1.0 times the gross registered tonnage.

The displacement of a ship is therefore the product of the length between perpendiculars (perps), the beam, the draft, the block coefficient and the density of the water. The block coefficient is the ratio between the volume of the wetted portion of the ship's hull (the displacement) and the volume of the enclosing block (length between the perps × the beam × the draft).

The displacement light varies from about 15–25 per cent of displacement fully loaded. The displacement light, i.e. the ship without ballast or any load, should for safety reasons only occur when the ship is moored in a dock or at a shipyard for fitting-out or repair.

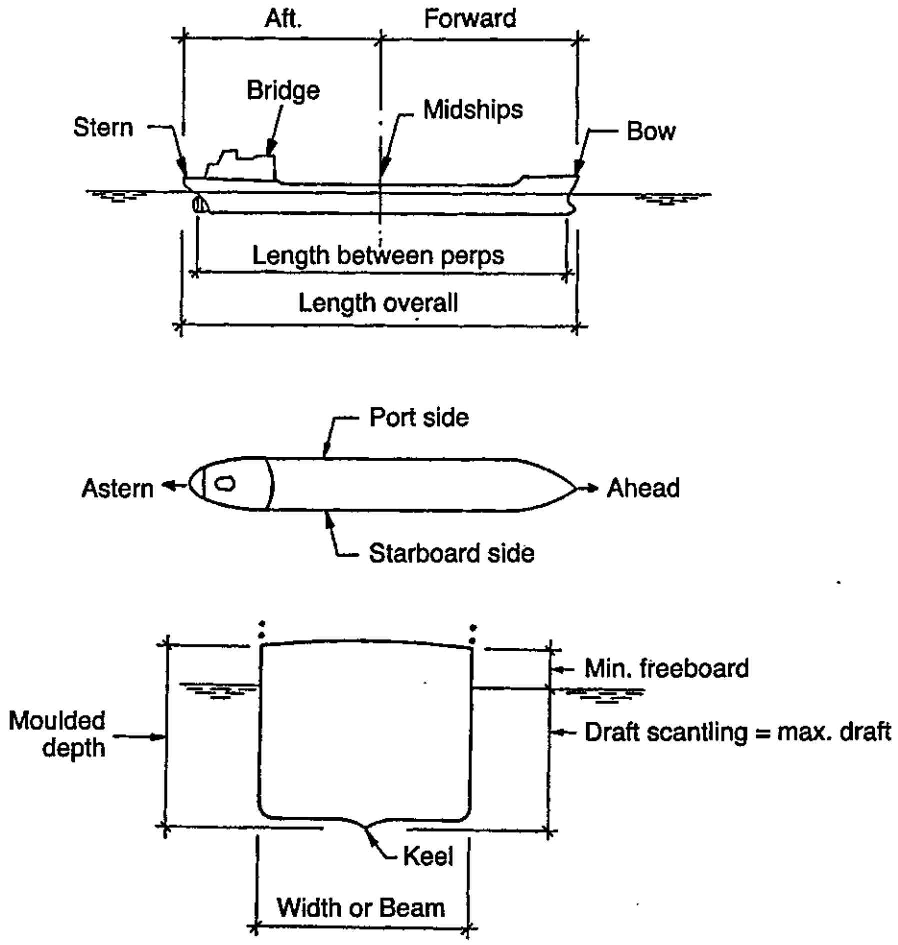

Fig. 20.1. Ship definitions

The ballast condition is the minimum weight or ballast a ship has to carry for safe manoeuvring stability. For example, having discharged all oil cargo, the oil tanker has to increase its weight by taking in seawater as ballast to increase the draft in order to obtain the necessary safe manoeuvring stability. The ballast displacement is about 30–50 per cent of the displacement fully loaded, depending on the weather conditions.

Air draft: the maximum distance from the water level to the highest point of the ship at the prevailing draft.

Scantling draft: the draft for which the structural strength of the ship has been designed.

Designed draft: the draft on which the fundamental design parameters of the ship are based.

Trim: the difference between the aft and the forward draft.

Bow to centre manifold/stern to centre manifold: the distance from the extreme point of bow or stern to the manifold centreline for tankers.

The definitions of ship overall dimensions, e.g. length, draft, etc., are illustrated in Fig. 20.1.

20.2 Ship dimensions

When the dimensions of the ships are not clearly known the average ship dimensions shown in the tables may be used in the design of berths, dolphins and fenders. The following tables show the general average dimensions for the beam, the overall length and the full loaded draft for general cargo ships, oil tankers and container ships. The length between perps is roughly 95 per cent of the length overall.

Passenger ships

GRT (ton)	DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Moulded depth (m)	Draft ballast loaded (m)	Draft max. (m)
80 000	–	75 000	315	295	35.5	25.0	–	11.5
70 000	–	65 000	312	295	34.0	24.0	–	11.0
60 000	–	55 000	310	290	32.5	23.0	–	10.5
50 000	–	45 000	300	280	31.0	21.0	–	10.0
40 000	–	35 000	265	245	29.5	18.0	–	10.0
30 000	–	30 000	230	210	28.0	17.0	–	10.0
20 000	–	20 000	200	180	25.0	15.0	–	9.2

Mixed cargo ships (*full deck construction*)

GRT (ton)	DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Moulded depth (m)	Draft ballast loaded (m)	Draft max. (m)
10 000	15 000	20 000	165	155	21.5	12.0	4.9	9.5
7000	10 000	14 000	145	135	20.0	11.5	4.4	8.5
5500	8000	11 000	135	125	18.0	10.5	4.1	8.0
4000	6000	8000	125	115	16.5	9.5	3.8	7.5
3500	5000	7000	105	100	15.0	8.5	3.6	7.0
2000	3000	4000	90	85	13.0	7.3	3.0	6.0
1300	2000	3000	80	75	12.0	6.5	2.6	5.3
1000	1500	2000	70	65	10.0	5.1	2.2	4.3
500	700	1000	55	50	8.5	4.5	1.9	3.8

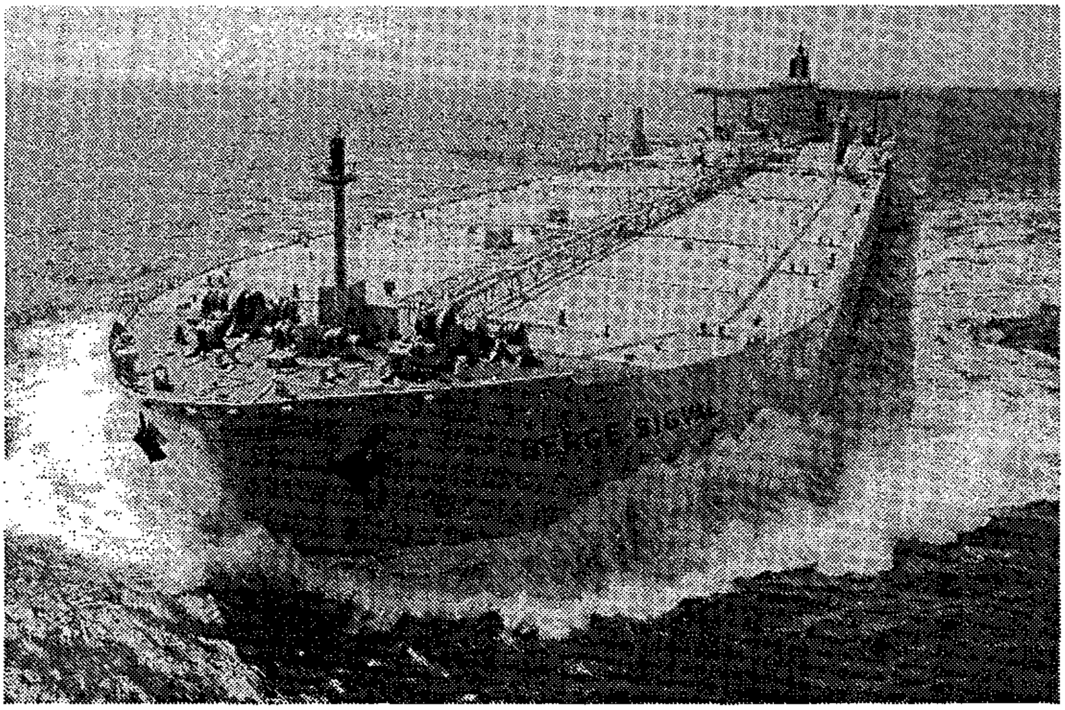

Fig. 20.2. Ultra-large crude carrier. Photo courtesy of Bergesen DY, Norway

Bulk cargo (*oil tankers, bulk carriers, etc.*)
The tankers are usually classified as shown below:

Type of tanker	Size in DWT
General purpose/product carrier	up to 25 000
Super tankers and large tankers	25 000–150 000
VLCC (very large crude carriers)	150 000–300 000
ULCC (ultra-large crude carriers) as shown in Fig. 20.2	more than 300 000

The bulk carriers are usually classified as shown below:

Type of bulk carrier	Size in DWT
Mini bulk carrier	<12 000
Small handy-sized	15 000–25 000
Handy-sized	25 000–50 000
Handy max	35 000–50 000
Panamax	50 000–80 000
Cape-sized	100 000–180 000
Very large bulk carrier	>180 000

DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Moulded depth (m)	Draft ballast loaded (m)	Draft max. (m)
450 000	525 000	425	404	68.5	31.0	13.0	25.0
400 000	460 000	412	392	66.0	29.0	12.5	24.0
300 000	356 000	385	364	59.5	27.0	12.0	22.0
275 000	326 000	376	355	57.5	26.5	11.8	21.5
250 000	300 000	367	346	55.5	26.0	11.5	21.0
225 000	270 000	356	336	53.5	25.5	11.0	20.5
200 000	240 000	345	326	51.0	25.0	10.5	19.5
175 000	212 000	330	315	48.5	24.0	10.0	18.5
150 000	180 000	315	300	46.0	23.0	9.0	17.0
125 000	155 000	295	280	43.5	22.0	8.5	16.0
100 000	125 000	280	265	41.0	21.0	8.0	15.0
85 000	105 000	265	255	38.0	19.0	7.5	14.0
70 000	85 000	255	245	35.5	18.5	7.2	13.5
50 000	60 000	225	215	32.0	16.5	6.4	12.0
30 000	37 000	195	185	27.0	14.2	5.8	11.0
25 000	30 000	185	180	25.0	13.5	5.5	10.5
15 000	20 000	165	155	21.0	12.0	4.9	9.5

Container ships

DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Draft max. (m)	Number of containers approx.	Generation
104 000	143 000	347	330	42.8	14.5	8000	7th
85 000	119 000	318	302	42.8	14.2	6000	6th
75 000	90 000	350	335	45.0	14.0	6000	6th
66 500	80 000	275	262	40.0	14.0	4800	5th
64 500	77 500	294	282	32.2	13.5	4400	5th
55 000	77 000	275	260	39.4	12.5	3900	4th
50 000	73 500	290	275	32.4	13.0	2800	3rd
42 000	61 000	285	270	32.3	12.0	2380	3rd
36 000	51 000	270	255	31.8	11.7	2000	3rd
30 000	41 500	228	214	31.0	11.3	1670	2nd
25 000	34 000	212	198	30.0	10.7	1380	2nd
20 000	27 000	198	184	28.7	10.0	1100	2nd
15 000	20 000	180	166	26.5	9.0	810	1st
10 000	13 500	159	144	23.5	8.0	530	1st
7000	9600	143	128	19.0	6.5	316	1st

Gas tankers

Capacity (m^3)	DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Moulded depth (m)	Draft max. (m)
LPG tankers							
75 000	46 900	75 000	229	218	36.0	21.0	12.1
52 000	38 500	53 200	206	196	31.4	18.6	11.3
35 000	36 200	7500	185	176	27.8	18.0	12.5
24 000	18 100	32 800	157	149	25.3	16.0	10.1
15 000	16 200	25 000	151	140	25.0	14.3	9.6
8300	9800	15 500	128	116	20.0	12.1	9.4
5000	5400	9000	106	98	17.0	10.0	7.4
2500	2800	5000	75	70	14.0	7.9	6.8
LNG tankers							
250 000	122 500	177 000	369	354	55.7	31.2	12.8
220 000	108 000	158 000	365	341	53.8	30.5	12.5
200 000	100 000	146 000	340	325	51.3	28.0	12.0
168 000	84 500	125 000	298	285	48.7	28.0	11.9
163 700	84 000	125 000	292	280	45.2	27.5	11.6
145 000	74 400	110 000	288	274	49.0	26.8	12.3
137 000	71 500	100 000	290	275	48.1	28.0	11.3
125 000	66 800	102 000	272	259	47.2	26.5	11.4
87 600	53 600	74 000	250	237	40.0	23.0	10.6
65 000	36 400	52 000	214	204	37.8	21.5	9.8
29 000	22 100	32 600	182	171	29.0	16.5	9.0
1000		1600	65		12.0	6.0	3.5

Ferries and ro/ro ships

DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Draft max. (m)
106 000	115 000	253	238	40.0	15.0
64 500	76 000	225	215	34.0	13.0
42 500	53 000	183	173	32.3	12.0
27 700	40 000	177	158	27.3	11.6
18 000	32 500	181	165	30.4	9.3
16 000	23 500	178	164	26.8	7.6
14 000	21 500	164	149	23.5	8.8
12 000	20 000	190	173	26.0	7.2
10 000	23 500	193	181	27.3	6.8
8000	16 000	156	137	22.6	7.3
6000	21 000	178	170	27.8	6.3

Car transport ships

DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Draft max. (m)	Number of cars approx.
28 000	45 000	198	183	32.3	11.8	6200
26 300	42 000	213	198	32.3	10.5	6000
17 900	33 000	195	180	32.3	9.7	5600

Fishing boats

GRT (ton)	DWT (ton)	Deplacement (ton)	Overall length (m)	Length between perps (m)	Beam (m)	Moulded depth (m)	Draft ballast loaded (m)	Draft max. (m)
2 500	–	2 800	90	80	14.0	–	–	5.9
2 000	–	2 500	85	75	13.0	–	–	5.6
1 500	–	2 100	80	70	12.0	–	–	5.3
1 000	–	1 750	75	65	11.0	–	–	5.0
800	–	1 550	70	60	10.5	–	–	4.8
600	–	1 200	65	55	10.0	–	–	4.5
400	–	800	55	45	8.5	–	–	4.0
200	–	400	40	35	7.0	–	–	3.5

General wind and current areas for different types of ship

In Fig. 20.3 the approximate laterally projected areas perpendicular to the wind direction above water of typical oil tankers, bulk and ore carriers and cargo ships in ballast and loaded condition relative to the ship's displacement are shown. For comparison the data shown in Figs 20.4 and 20.5 are taken from the British Standard and from research done by The Port and Harbour Research Institute, Ministry of Transport, Japan. The values given in figures must only be used as a rough indicator for the ship's dimensions. For important calculations the actual dimensions of the ships that will call at the berth or harbour, should be used.

Figure 20.5 shows data from British Standard BS 6349, Part 1 for the length and the laterally projected areas for container ships in loaded and ballasted conditions, compared to general approximately lateral projected wind area's data for container ships with and without containers on the deck shown in the table below the figure.

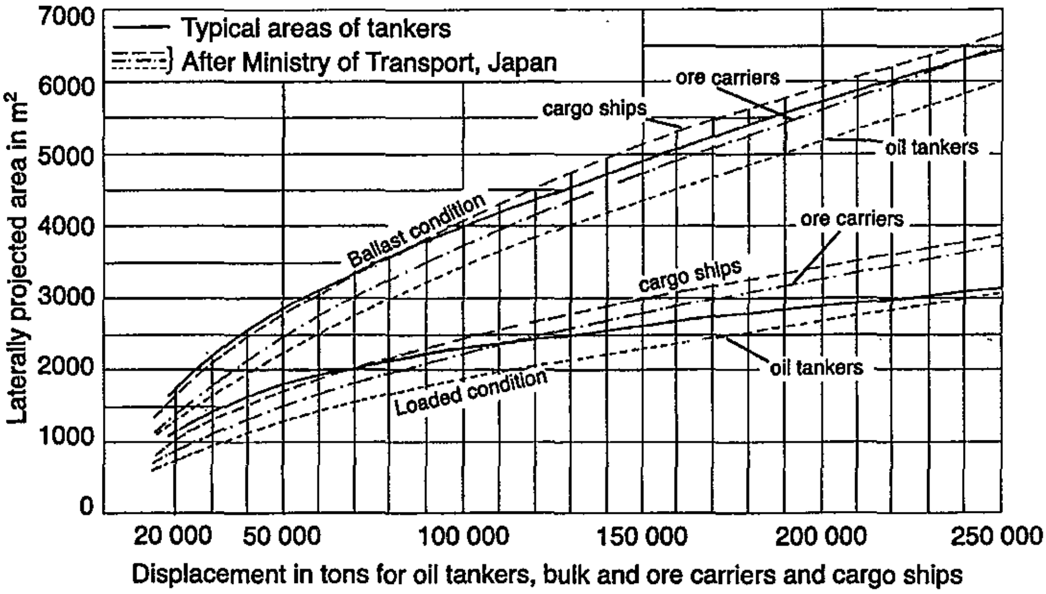

Fig. 20.3. Laterally projected wind areas of ships relative to displacements

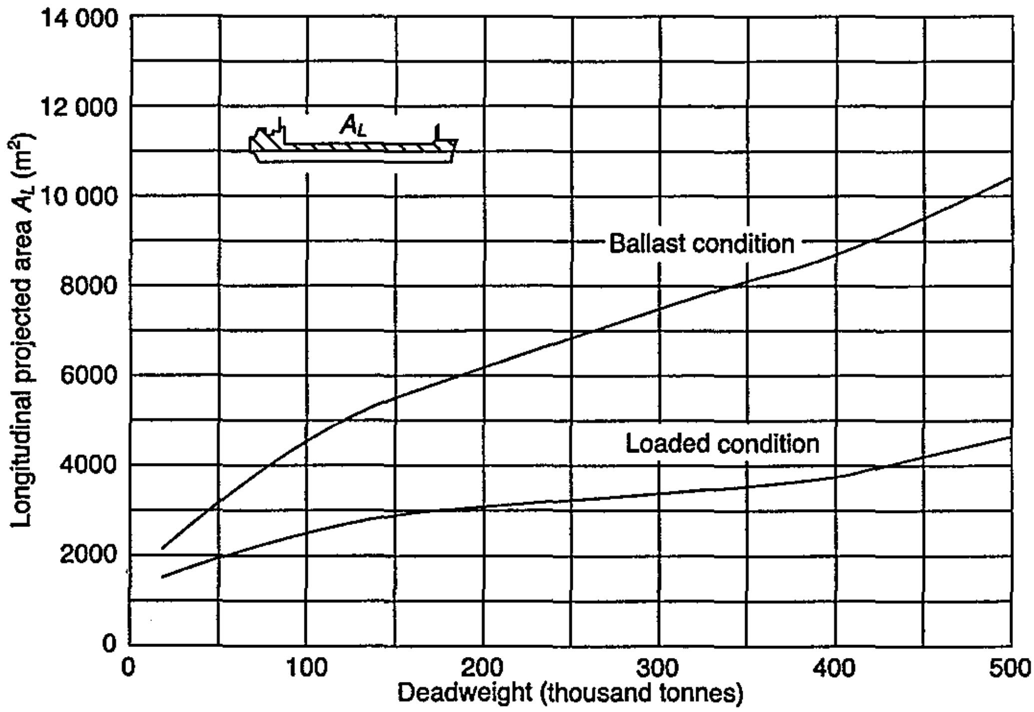

Fig. 20.4. Typical longitudinal projected areas of tankers relative to deadweight

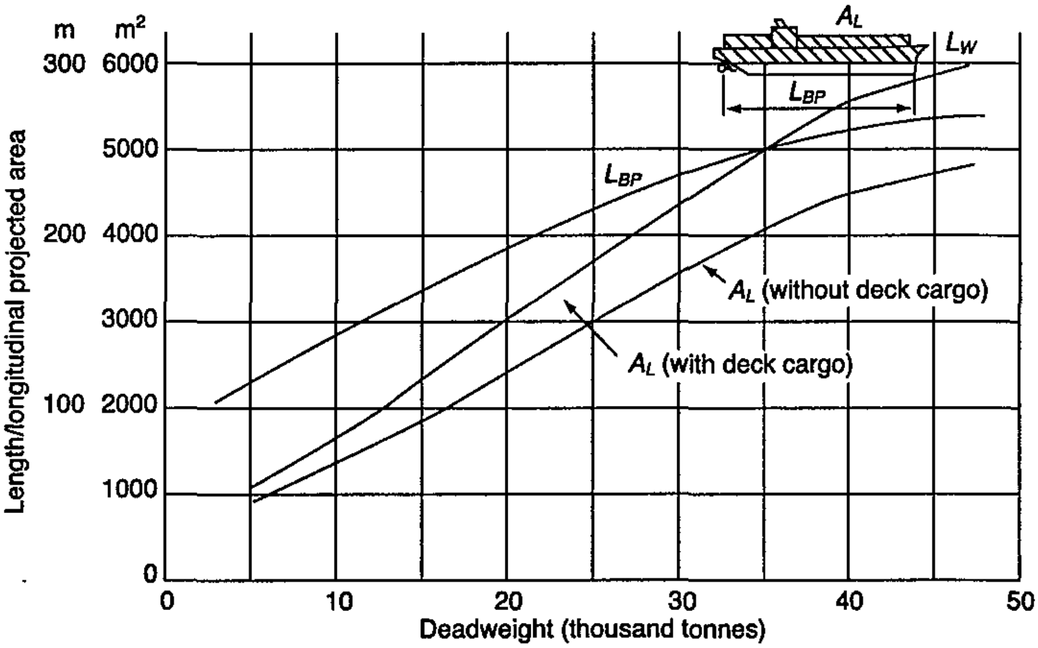

Fig. 20.5. Length and longitudinal projected areas of container ships relative to deadweight

Deadweight	Container ship wind area in m³	
	Without containers on deck	With containers on deck
50 000	4900	6100
42 000	4600	5700
30 000	3600	4400
25 000	3000	3600
20 000	2400	3100
15 000	1900	2300
10 000	1400	1700

The approximately lateral projected wind and current areas above and below sea level for some LPG and LNG tankers in fully and ballast loaded condition are shown in the following table.

	Above sea level				Below sea level			
	Fully loaded (m^2)		Ballast loaded (m^2)		Fully loaded (m^2)		Ballast loaded (m^2)	
	Lateral	Front	Lateral	Front	Lateral	Front	Lateral	Front
LPG tankers								
75 000 m^3	2400	760	3640	960	2970	490	1760	280
52 000 m^3	2100	630	3000	770	2450	370	1560	240
24 000 m^3	1000	320	1850	460	1790	300	920	160
LNG tankers								
220 000 m^3	11 500	2200	13 000	2400	4400	670	2800	430
200 000 m^3	9700	2000	11 200	2300	4000	610	2500	390
168 000 m^3	8700	2300	9800	2500	3300	600	2400	400
145 000 m^3	7600	1950	8400	2100	3400	600	2500	450
137 000 m^3	7000	1800	8400	2100	2300	550	2100	360
125 000 m^3	7320	1600	8430	2000	3190	530	2120	350
87 600 m^3	5900	1200	6780	1400	2490	420	1660	280
65 000 m^3	4300	1100	4900	1200	2000	370	1380	250
29 000 m^3	3160	900	3700	990	1540	260	1090	180

20.3 Recommended design dimensions

The Port and Harbour Research Institute, Ministry of Transport, Japan has given the following formulas, as shown in Fig. 20.6, for the correlation between the displacement tonnage (DT), the laterally projected area, the front area, the total surface area below sea level, the displacement ballast loaded, the draft ballast loaded, etc. for general cargo, oil tankers and ore carriers.

As a useful rule of thumb, the following approximate formula gives the full loaded draft of cargo ships and bulk carriers in m:

$$\text{Full loaded draft} = \sqrt{\frac{\text{DWT}}{1000}} + 5$$

The formula will give the full loaded draft to within 1 m for general cargo ships and dry- and liquid-bulk carriers over the range 5000 to 400 000 DWT.

As an example of the use of Fig. 20.6 the following approximate values can be used as a rough indicator for a 50 000 DWT oil tanker:

$$\text{Displacement fully loaded} = 2.028 \times 50\,000^{0.954} = 61\,643 \text{ ton}$$

Type of ship				General cargos		Oil tankers		Ore carriers	
Range of tonnage in DWT				500–140 000		500–320 000		500–200 000	
Coefficients				α	β	α	β	α	β
Displacement fully loaded = DT				2.463	0.936	2.028	0.954	1.687	0.969
Laterally projected area	Above sea level	Fully loaded	$= \alpha(DWT)^\beta$	8.770	0.496	4.964	0.522	4.390	0.548
Laterally projected area	Above sea level	Ballast loaded	$= \alpha(DWT)^\beta$	9.641	0.533	5.943	0.562	5.171	0.580
Laterally projected area	Below sea level	Fully loaded	$= \alpha(DWT)^\beta$	3.495	0.608	3.198	0.611	2.723	0.625
Laterally projected area	Below sea level	Ballast loaded	$= \alpha(DWT)^\beta$	1.404	0.627	1.629	0.610	1.351	0.633
Front area	Above sea level	Fully loaded	$= \alpha(DWT)^\beta$	2.763	0.490	2.666	0.478	1.971	0.510
Front area	Above sea level	Ballast loaded	$= \alpha(DWT)^\beta$	3.017	0.510	2.485	0.517	1.967	0.538
Total surface area below sea level		Fully loaded	$= \alpha(DWT)^\beta$	9.260	0.639	6.162	0.673	4.576	0.702
Total surface area below sea level		Ballast loaded	$= \alpha(DWT)^\beta$	4.637	0.669	3.865	0.686	3.471	0.701
Displacement ballast loaded			$= \alpha(DT)^\beta$	0.199	1.084	0.383	1.018	0.385	1.023
Draft ballast loaded			$= \alpha(draft_{max})^\beta$	0.352	1.172	0.548	0.966	0.551	0.993

Fig. 20.6. Laterally projected wind area of ships relative to ships displacements

Laterally projected area above sea level fully loaded

$$= 4.964 \times 50\,000^{0.522} = 1408\,\text{m}^2$$

Displacement ballast loaded $= 0.383 \times 61\,643^{1.018} = 28\,792$ ton

Maximum draft after table $= 12.0$ m

$$\text{Maximum draft} = \sqrt{\frac{50000}{1000}} + 5 = 12.07\,\text{m}$$

Draft ballast loaded $= 0.548 \times 12.07^{0.966} = 6.08$ m

Draft minimum after IMO $= 0.02 \times L_{bp} + 2.0 = 6.3$ m

Draft ballast after table $= 6.4$ m

When designing harbours and berth facilities including the fender design, the design vessel (DV) should be the largest ship expected to berth. The data in the tables below are by Akakura and Takahashi, Technical Note of the Ports and Harbour Research Institute No. 911, September 1998 and Port and Harbour Bureau of Ministry of Transport, Japan. The tables below show the 50, 75, 90 and 95 per cent confidence limits. The L_{OA} is the ship length overall, L_{BP} is the length between perps, B is the beam and D is the moulded depth.

Confidence limit: 50 per cent

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
General cargo ship	1000	1580	63	58	10.3	5.2	3.6	227	292	59	88
	2000	3040	78	72	12.4	6.4	4.5	348	463	94	134
	3000	4460	88	82	13.9	7.2	5.1	447	605	123	172
	5000	7210	104	96	16.0	8.4	6.1	612	849	173	236
	7000	9900	115	107	17.6	9.3	6.8	754	1060	216	290
	10 000	13 900	128	120	19.5	10.3	7.6	940	1340	274	361
	15 000	20 300	146	136	21.8	11.7	8.7	1210	1760	359	463
	20 000	26 600	159	149	23.6	12.7	9.6	1440	2130	435	552
	30 000	39 000	181	170	26.4	14.4	10.9	1850	2780	569	709
	40 000	51 100	197	186	28.6	15.7	12.0	2210	3370	690	846
Bulk carrier*	5000	6740	106	98	15.0	8.4	6.1	615	850	205	231
	7000	9270	116	108	16.6	9.3	6.7	710	1010	232	271
	10 000	13 000	129	120	18.5	10.4	7.5	830	1230	264	320
	15 000	19 100	145	135	21.0	11.7	8.4	980	1520	307	387
	20 000	25 000	157	148	23.0	12.8	9.2	1110	1770	341	443
	30 000	36 700	176	167	26.1	14.4	10.3	1320	2190	397	536
	50 000	59 600	204	194	32.3	16.8	12.0	1640	2870	479	682
	70 000	81 900	224	215	32.3	18.6	13.3	1890	3440	542	798
	100 000	115 000	248	239	37.9	20.7	14.8	2200	4150	619	940
	150 000	168 000	279	270	43.0	23.3	16.7	2610	5140	719	1140
	200 000	221 000	303	294	47.0	25.4	18.2	2950	5990	800	1310
	250 000	273 000	322	314	50.4	27.2	19.4	3240	6740	868	1450

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Container ship**	7000	10 200	116	108	19.6	9.3	6.9	1320	1360	300	396
	10 000	14 300	134	125	21.6	10.7	7.7	1690	1700	373	477
	15 000	21 100	157	147	24.1	12.6	8.7	2250	2190	478	591
	20 000	27 800	176	165	26.1	14.1	9.5	2750	2620	569	687
	25 000	34 300	192	180	27.7	15.4	10.2	3220	3010	652	770
	30 000	40 800	206	194	29.1	16.5	10.7	3660	3370	729	850
	40 000	53 700	231	218	32.3	18.5	11.7	4480	4040	870	990
	50 000	66 500	252	238	32.3	20.2	12.5	5230	4640	990	1110
	60 000	79 100	271	256	35.2	21.7	13.2	5950	5200	1110	1220
Oil tanker	1000	1450	59	54	9.7	4.3	3.8	170	266	78	80
	2000	2810	73	68	12.1	5.4	4.7	251	401	108	117
	3000	4140	83	77	13.7	6.3	5.3	315	509	131	146
	5000	6740	97	91	16.0	7.5	6.1	419	689	167	194
	7000	9300	108	102	17.8	8.4	6.7	505	841	196	233
	10 000	13 100	121	114	19.9	9.5	7.5	617	1040	232	284
	15 000	19 200	138	130	22.5	11.0	8.4	770	1320	281	355
	20 000	25 300	151	143	24.6	12.2	9.1	910	1560	322	416
	30 000	37 300	171	163	27.9	14.0	10.3	1140	1990	390	520
	50 000	60 800	201	192	32.3	16.8	11.9	1510	2690	497	689
	70 000	83 900	224	214	36.3	18.9	13.2	1830	3280	583	829
	100 000	118 000	250	240	40.6	21.4	14.6	2230	4050	690	1010
	150 000	174 000	284	273	46.0	24.7	16.4	2800	5150	840	1260
	200 000	229 000	311	300	50.3	27.3	17.9	3290	6110	960	1480
	300 000	337 000	354	342	57.0	31.5	20.1	4120	7770	1160	1850

Ro/ro ship	1000	1970	66	60	13.2	5.2	3.2	700	810	216	217
	2000	3730	85	78	15.6	7.0	4.1	970	1110	292	301
	3000	5430	99	90	17.2	8.4	4.8	1170	1340	348	364
	5000	8710	119	109	19.5	10.5	5.8	1480	1690	435	464
	7000	11 900	135	123	21.2	12.1	6.6	1730	1970	503	544
	10 000	16 500	153	141	23.1	14.2	7.5	2040	2320	587	643
	15 000	24 000	178	163	25.6	16.9	8.7	2460	2790	701	779
	20 000	31 300	198	182	27.4	19.2	9.7	2810	3180	794	890
	30 000	45 600	229	211	30.3	23.0	11.3	3400	3820	950	1080

Notes

* Full-load condition of wind lateral/front areas of log carrier do not include the areas of logs on deck

** Full-load condition of wind lateral/front areas of container ships include the areas of containers on deck

Type	Gross tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Passenger ship	1000	850	60	54	11.4	4.1	1.9	426	452	167	175
	2000	1580	76	68	13.6	5.3	2.5	683	717	225	234
	3000	2270	87	78	15.1	6.2	3.0	900	940	267	277
	5000	3580	104	92	17.1	7.5	3.6	1270	1320	332	344
	7000	4830	117	103	18.6	8.6	4.1	1600	1650	383	396
	10 000	6640	133	116	20.4	9.8	4.8	2040	2090	446	459
	15 000	9530	153	132	22.5	11.5	5.6	2690	2740	530	545
	20 000	12 300	169	146	24.2	12.8	7.6	3270	3320	599	614
	30 000	17 700	194	166	26.8	14.9	7.6	4310	4350	712	728
	50 000	27 900	231	197	30.5	18.2	7.6	6090	6120	880	900
	70 000	37 600	260	220	33.1	20.7	7.6	7660	7660	1020	1040
Ferry	1000	810	59	54	12.7	4.6	2.7	387	404	141	145
	2000	1600	76	69	15.1	5.8	3.3	617	646	196	203
	3000	2390	88	80	16.7	6.5	3.7	811	851	237	247
	5000	3940	106	97	19.0	7.6	4.3	1150	1200	302	316
	7000	5480	119	110	20.6	8.5	4.8	1440	1510	354	372
	10 000	7770	135	125	22.6	9.5	5.3	1830	1930	419	442
	15 000	11 600	157	145	25.0	10.7	6.0	2400	2540	508	537
	20 000	15 300	174	162	26.8	11.7	6.5	2920	3090	582	618
	30 000	22 800	201	188	29.7	13.3	7.4	3830	4070	705	752
	40 000	30 300	223	209	31.9	14.5	8.0	4660	4940	810	860

Gas carrier	1000	2210	68	63	11.1	5.3	4.3	350	436	121	139
	2000	4080	84	78	13.7	6.8	5.2	535	662	177	203
	3000	5830	95	89	15.4	7.8	5.8	686	846	222	254
	5000	9100	112	104	17.9	9.4	6.7	940	1150	295	335
	7000	12 300	124	116	19.8	10.6	7.4	1150	1410	355	403
	10 000	16 900	138	130	22.0	12.0	8.2	1430	1750	432	490
	15 000	24 100	157	147	24.8	13.9	9.3	1840	2240	541	612
	20 000	31 100	171	161	27.1	15.4	10.0	2190	2660	634	716
	30 000	44 400	194	183	30.5	17.8	11.7	2810	3400	794	894
	50 000	69 700	227	216	35.5	21.3	11.7	3850	4630	1050	1180
	70 000	94 000	252	240	39.3	24.0	11.7	4730	5670	1270	1420
	100 000	128 000	282	268	43.7	27.3	11.7	5880	7030	1550	1730

Confidence limit: 75 per cent

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
General cargo ship	1000	1690	67	62	10.8	5.8	3.9	278	342	63	93
	2000	3250	83	77	13.1	7.2	4.9	426	541	101	142
	3000	4750	95	88	14.7	8.1	5.6	547	708	132	182
	5000	7690	111	104	16.9	9.4	6.6	750	993	185	249
	7000	10 600	123	115	18.6	10.4	7.4	922	1240	232	307
	10 000	14 800	137	129	20.5	11.6	8.3	1150	1570	294	382
	15 000	21 600	156	147	23.0	13.1	9.5	1480	2060	385	490
	20 000	28 400	170	161	24.9	14.3	10.4	1760	2490	466	585
	30 000	41 600	193	183	27.8	16.2	11.9	2260	3250	611	750
	40 000	54 500	211	200	30.2	17.6	13.0	2700	3940	740	895
Bulk carrier*	5000	6920	109	101	15.5	8.6	6.2	689	910	221	245
	7000	9520	120	111	17.2	9.5	6.9	795	1090	250	287
	10 000	13 300	132	124	19.2	10.6	7.7	930	1320	286	340
	15 000	19 600	149	140	21.8	11.9	8.6	1100	1630	332	411
	20 000	25 700	161	152	23.8	13.0	9.4	1240	1900	369	470
	30 000	37 700	181	172	27.0	14.7	10.6	1480	2360	428	569
	50 000	61 100	209	200	32.3	17.1	12.4	1830	3090	518	723
	70 000	84 000	231	221	32.3	18.9	13.7	2110	3690	586	846
	100 000	118 000	255	246	39.2	21.1	15.2	2460	4460	669	1000
	150 000	173 000	287	278	44.5	23.8	17.1	2920	5520	777	1210
	200 000	227 000	311	303	48.7	25.9	18.6	3300	6430	864	1380
	250 000	280 000	332	324	52.2	27.7	19.9	3630	7240	938	1540
Container ship**	7000	10 700	123	115	20.3	9.8	7.2	1460	1590	330	444
	10 000	15 100	141	132	22.4	11.3	8.0	1880	1990	410	535
	15 000	22 200	166	156	25.0	13.3	9.0	2490	2560	524	663
	20 000	29 200	186	175	27.1	14.9	9.9	3050	3070	625	771

	25 000	36 100	203	191	28.8	16.3	10.6	3570	3520	716	870
	30 000	43 000	218	205	30.2	17.5	11.1	4060	3950	800	950
	40 000	56 500	244	231	32.3	19.6	12.2	4970	4730	950	1110
	50 000	69 900	266	252	32.3	21.4	13.0	5810	5430	1090	1250
	60 000	83 200	286	271	36.5	23.0	13.8	6610	6090	1220	1370
Oil tanker	1000	1580	61	58	10.2	4.5	4.0	190	280	86	85
	2000	3070	76	72	12.6	5.7	4.9	280	422	119	125
	3000	4520	87	82	14.3	6.6	5.5	351	536	144	156
	5000	7360	102	97	16.8	7.9	6.4	467	726	184	207
	7000	10 200	114	108	18.6	8.9	7.1	564	885	216	249
	10 000	14 300	127	121	20.8	10.0	7.9	688	1090	255	303
	15 000	21 000	144	138	23.6	11.6	8.9	860	1390	309	378
	20 000	27 700	158	151	25.8	12.8	9.6	1010	1650	355	443
	30 000	40 800	180	173	29.2	14.8	10.9	1270	2090	430	554
	50 000	66 400	211	204	32.3	17.6	12.6	1690	2830	548	734
	70 000	91 600	235	227	38.0	19.9	13.9	2040	3460	642	884
	100 000	129 000	263	254	42.5	22.5	15.4	2490	4270	761	1080
	150 000	190 000	298	290	48.1	25.9	17.4	3120	5430	920	1340
	200 000	250 000	327	318	52.6	28.7	18.9	3670	6430	1060	1570
	300 000	368 000	371	363	59.7	33.1	21.2	4600	8180	1280	1970
Ro/ro ship	1000	2190	73	66	14.0	6.2	3.5	880	970	232	232
	2000	4150	94	86	16.6	8.4	4.5	1210	1320	314	323
	3000	6030	109	99	18.3	10.0	5.3	1460	1590	374	391
	5000	9670	131	120	20.7	12.5	6.4	1850	2010	467	497
	7000	13 200	148	136	22.5	14.5	7.2	2170	2350	541	583
	10 000	18 300	169	155	24.6	17.0	8.2	2560	2760	632	690
	15 000	26 700	196	180	27.2	20.3	9.6	3090	3320	754	836
	20 000	34 800	218	201	29.1	23.1	10.7	3530	3780	854	960
	30 000	50 600	252	233	32.2	27.6	12.4	4260	4550	1020	1160

Notes
* Full-load condition of wind lateral/front areas of log carrier do not include the areas of logs on deck
** Full-load condition of wind lateral/front areas of container ships include the areas of containers on deck

Type	Gross tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Passenger ship	1000	1030	64	60	12.1	4.9	2.6	464	486	187	197
	2000	1910	81	75	14.4	6.3	3.4	744	770	251	263
	3000	2740	93	86	16.0	7.4	4.0	980	1010	298	311
	5000	4320	112	102	18.2	9.0	4.8	1390	1420	371	386
	7000	5830	125	114	19.8	10.2	5.5	1740	1780	428	444
	10 000	8010	142	128	21.6	11.7	6.4	2220	2250	498	516
	15 000	11 500	163	146	23.9	13.7	7.5	2930	2950	592	611
	20 000	14 900	180	160	25.7	15.3	8.0	3560	3570	669	690
	30 000	21 300	207	183	28.4	17.8	8.0	4690	4680	795	818
	50 000	33 600	248	217	32.3	21.7	8.0	6640	6580	990	1010
	70 000	45 300	278	243	35.2	24.6	8.0	8350	8230	1140	1170
Ferry	1000	1230		61	14.3	5.5	3.4	411	428	154	158
	2000	2430	86	78	17.0	6.8	4.2	656	685	214	221
	3000	3620	99	91	18.8	7.7	4.8	862	903	259	269
	5000	5970	119	110	21.4	9.0	5.5	1220	1280	330	344
	7000	8310	134	124	23.2	10.0	6.1	1530	1600	387	405
	10 000	11 800	153	142	25.4	11.1	6.8	1940	2040	458	482
	15 000	17 500	177	164	28.1	12.6	7.6	2550	2690	555	586
	20 000	23 300	196	183	30.2	13.8	8.3	3100	3270	636	673
	30 000	34 600	227	212	33.4	15.6	9.4	4070	4310	771	819
	40 000	45 900	252	236	35.9	17.1	10.2	4950	5240	880	940

Gas	1000	2480	71	66	11.7	5.7	4.6	390	465	133	150
carrier	2000	4560	88	82	14.3	7.2	5.7	597	707	195	219
	3000	6530	100	93	16.1	8.4	6.4	765	903	244	273
	5000	10 200	117	109	18.8	10.0	7.4	1050	1230	323	361
	7000	13 800	129	121	20.8	11.3	8.1	1290	1510	389	434
	10 000	18 900	144	136	23.1	12.9	9.0	1600	1870	474	527
	15 000	27 000	164	154	26.0	14.9	10.1	2050	2390	593	658
	20 000	34 800	179	169	28.4	16.5	11.0	2450	2840	696	770
	30 000	49 700	203	192	32.0	19.0	12.3	3140	3630	870	961
	50 000	78 000	237	226	37.2	22.8	12.3	4290	4940	1150	1270
	70 000	105 000	263	251	41.2	25.7	12.3	5270	6050	1390	1530
	100 000	144 000	294	281	45.8	29.2	12.3	6560	7510	1690	1860

Confidence limit: 90 per cent

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
General cargo ship	1000	1790	72	66	11.4	6.5	4.2	333	394	67	98
	2000	3440	89	83	13.8	8.0	5.3	511	623	107	149
	3000	5040	101	94	15.4	9.0	6.0	656	815	140	192
	5000	8150	118	111	17.8	10.5	7.1	899	1143	197	262
	7000	11 200	131	123	19.5	11.6	8.0	1106	1430	247	323
	10 000	15 700	146	138	21.5	12.9	8.9	1380	1810	313	402
	15 000	22 900	166	157	24.1	14.6	10.2	1770	2370	410	516
	20 000	30 100	181	172	26.1	15.9	11.2	2110	2860	496	615
	30 000	44 000	205	195	29.2	18.0	12.8	2710	3740	650	789
	40 000	57 700	224	214	31.6	19.6	14.0	3240	4530	788	942
Bulk carrier*	5000	7090	111	103	16.0	8.7	6.4	763	970	237	259
	7000	9740	123	114	17.7	9.7	7.1	880	1160	268	303
	10 000	13 700	136	127	19.8	10.8	7.9	1020	1400	306	358
	15 000	20 000	152	143	22.5	12.1	8.9	1220	1740	356	433
	20 000	26 300	165	156	24.6	13.2	9.6	1370	2030	395	495
	30 000	38 600	186	176	27.9	14.9	10.9	1630	2510	459	599
	50 000	62 600	215	206	32.3	17.4	12.7	2030	3290	555	761
	70 000	86 000	236	227	32.3	19.3	14.0	2340	3930	628	892
	100 000	121 000	262	253	40.5	21.4	15.5	2720	4750	717	1050
	150 000	177 000	294	286	45.9	24.2	17.5	3240	5890	833	1280
	200 000	232 000	319	311	50.2	26.4	19.1	3660	6860	926	1460
	250 000	287 000	340	333	53.8	28.2	20.4	4020	7720	1006	1620

Container ship**	7000	11 200	129	121	21.1	10.3	7.4	1600	1830	358	492
	10 000	15 800	148	139	23.2	11.9	8.3	2060	2290	445	594
	15 000	23 200	174	164	25.9	14.0	9.3	2740	2950	570	735
	20 000	30 500	195	184	28.0	15.7	10.2	3360	3530	679	855
	25 000	37 800	213	201	29.8	17.1	10.9	3930	4060	778	960
	30 000	45 000	229	216	31.3	18.4	11.5	4460	4550	869	1060
	40 000	59 100	256	243	32.3	20.6	12.6	5460	5450	1040	1232
	50 000	73 200	280	266	32.3	22.5	13.5	6390	6260	1190	1380
	60 000	87 100	301	286	37.8	24.2	14.2	7260	7020	1330	1520
Oil tanker	1000	1710	64	61	10.6	4.7	4.2	210	293	94	90
	2000	3320	80	76	13.1	6.0	5.2	309	442	130	132
	3000	4890	91	87	14.9	6.9	5.8	388	562	158	165
	5000	7970	107	102	17.5	8.2	6.8	516	760	201	219
	7000	11 000	119	114	19.4	9.3	7.5	623	928	235	263
	10 000	15 500	133	128	21.6	10.5	8.3	760	1150	279	320
	15 000	22 800	151	146	24.5	12.1	9.3	950	1460	338	401
	20 000	30 000	165	160	26.8	13.4	10.1	1120	1730	387	469
	30 000	44 200	188	182	30.4	15.4	11.4	1400	2190	469	587
	50 000	72 000	220	215	32.3	18.5	13.2	1870	2970	598	777
	70 000	99 200	245	239	39.6	20.8	14.6	2250	3620	701	935
	100 000	140 000	274	268	44.2	23.5	16.2	2750	4470	830	1140
	150 000	206 000	312	306	50.2	27.1	18.2	3450	5690	1010	1420
	200 000	271 000	341	336	54.8	30.0	19.8	4050	6740	1150	1670
	300 000	399 000	388	382	62.2	34.6	22.3	5080	8570	1400	2080

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Ro/ro ship	1000	2400	79	72	14.8	7.3	3.8	1080	1130	248	248
	2000	4560	102	94	17.5	9.9	4.9	1480	1550	335	344
	3000	6630	118	109	19.3	11.8	5.8	1790	1860	400	416
	5000	10 620	143	131	21.9	14.8	7.0	2270	2350	499	530
	7000	14 500	161	149	23.8	17.1	7.9	2650	2740	578	621
	10 000	20 200	184	170	26.0	20.0	9.0	3130	3230	675	736
	15 000	29 300	213	197	28.7	23.9	10.5	3780	3880	805	891
	20 000	38 200	237	219	30.8	27.2	11.6	4320	4430	912	1020
	30 000	55 600	275	255	34.0	32.5	13.5	5210	5330	1090	1240

Notes
* Full-load condition of wind lateral/front areas of log carrier do not include the areas of logs on deck
** Full-load condition of wind lateral/front areas of container ships include the areas of containers on deck

Type	Gross tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Passenger ship	1000	1220	68	65	12.8	5.7	3.3	502	518	207	218
	2000	2260	86	82	15.2	7.4	4.4	804	822	278	292
	3000	3240	99	94	16.9	8.7	5.1	1060	1080	330	346
	5000	5110	119	111	19.2	10.5	6.3	1500	1510	410	428
	7000	6900	133	124	20.8	12.0	7.2	1880	1890	473	493
	10 000	9480	151	139	22.8	13.7	8.2	2400	2400	551	573
	15 000	13 600	173	159	25.2	16.0	8.4	3160	3150	654	679
	20 000	17 600	192	175	27.1	17.9	8.4	3850	3810	740	766
	30 000	25 200	220	200	30.0	20.9	8.4	5070	4990	879	907
	50 000	39 700	263	237	34.1	25.4	8.4	7170	7020	1090	1120
	70 000	53 600	296	265	37.1	28.9	8.4	9020	8780	1260	1290
Ferry	1000	1790	74	68	15.9	6.3	4.3	434	451	167	171
	2000	3540	95	87	18.9	7.8	5.3	693	722	232	239
	3000	5260	110	101	20.9	8.8	5.9	911	951	281	291
	5000	8690	133	122	23.8	10.4	6.9	1290	1350	358	372
	7000	12 100	150	139	25.9	11.5	7.6	1610	1690	420	438
	10 000	17 100	170	158	28.3	12.8	8.4	2050	2150	497	521
	15 000	25 500	197	184	31.3	14.5	9.5	2700	2840	602	633
	20 000	33 800	219	204	33.6	15.9	10.3	3270	3450	690	728
	30 000	50 300	253	237	37.2	18.0	11.7	4300	4540	836	886
	40 000	66 800	281	264	39.9	19.7	12.7	5230	5520	960	1020

Type	Gross tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Gas carrier	1000	2740	74	68	12.2	6.0	5.0	431	493	144	160
	2000	5050	91	85	14.9	7.7	6.1	659	750	211	233
	3000	7230	104	97	16.8	8.9	6.9	845	958	265	291
	5000	11 300	121	114	19.6	10.7	8.0	1160	1300	351	385
	7000	15 300	135	126	21.7	12.0	8.8	1420	1600	423	463
	10 000	20 900	150	141	24.1	13.7	9.8	1770	1980	515	563
	15 000	29 900	170	161	27.2	15.8	11.0	2260	2530	645	702
	20 000	38 500	186	176	29.6	17.5	11.9	2700	3010	756	822
	30 000	55 100	211	200	33.4	20.2	12.8	3460	3850	946	1026
	50 000	86 400	247	235	38.9	24.3	12.8	4740	5240	1250	1360
	70 000	116 000	274	262	42.9	27.4	12.8	5820	6420	1510	1630
	100 000	159 000	306	293	47.7	31.1	12.8	7240	7960	1840	1980

Confidence limit: 95 per cent

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
General cargo ship	1000	1850	74	69	11.7	6.9	4.4	372	428	70	101
	2000	3560	92	86	14.2	8.5	5.5	570	678	111	154
	3000	5210	104	98	15.9	9.6	6.3	732	887	146	198
	5000	8440	122	115	18.3	11.2	7.5	1003	1243	205	271
	7000	11 600	136	128	20.1	12.4	8.3	1234	1550	256	333
	10 000	16 200	151	143	22.2	13.8	9.3	1540	1970	325	414
	15 000	23 700	172	163	24.8	15.6	10.7	1970	2570	426	532
	20 000	31 100	188	179	26.9	17.0	11.7	2360	3110	516	634
	30 000	45 600	213	203	30.1	19.2	13.4	3030	4070	675	814
	40 000	59 800	233	223	32.6	20.9	14.7	3610	4930	818	971
Bulk carrier*	5000	7190	113	105	16.3	8.8	6.5	811	1010	247	267
	7000	9880	124	116	18.1	9.8	7.2	936	1210	280	312
	10 000	13 800	138	129	20.2	10.9	8.0	1090	1460	319	369
	15 000	20 300	155	146	22.9	12.3	9.0	1290	1810	371	447
	20 000	26 700	168	159	25.0	13.4	9.8	1460	2110	412	511
	30 000	39 100	188	179	28.4	15.1	11.0	1740	2610	479	618
	50 000	63 500	218	209	32.3	17.6	12.8	2160	3420	578	786
	70 000	87 200	240	231	32.3	19.5	14.2	2490	4090	655	920
	100 000	122 000	266	257	41.2	21.6	15.8	2890	4940	747	1090
	150 000	179 000	298	290	46.8	24.4	17.8	3440	6120	868	1320
	200 000	236 000	324	316	51.1	26.6	19.4	3890	7130	965	1510
	250 000	291 000	345	338	54.8	28.5	20.7	4270	8020	1048	1670

Type	Dead weight tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Container ship**	7000	11 500	133	125	21.5	10.6	7.6	1700	2000	377	524
	10 000	16 200	153	144	23.7	12.3	8.4	2180	2490	468	632
	15 000	23 900	179	169	26.4	14.4	9.5	2900	3210	599	782
	20 000	31 400	201	190	28.6	16.1	10.4	3550	3850	714	910
	25 000	38 800	219	208	30.4	17.6	11.1	4150	4420	818	1020
	30 000	46 200	236	223	31.9	18.9	11.8	4720	4950	914	1130
	40 000	60 800	264	251	32.3	21.2	12.8	5780	5930	1090	1310
	50 000	75 200	288	274	32.3	23.2	13.7	6760	6820	1250	1470
	60 000	89 400	310	295	38.5	24.9	14.5	7680	7640	1390	1620
Oil tanker	1000	1800	66	63	10.9	4.8	4.4	223	302	99	93
	2000	3480	82	78	13.5	6.1	5.3	328	455	137	137
	3000	5130	93	89	15.3	7.1	6.0	412	578	166	171
	5000	8360	109	105	17.9	8.5	7.0	548	782	211	226
	7000	11 500	122	118	19.9	9.5	7.7	661	954	248	272
	10 000	16 200	136	132	22.2	10.8	8.5	806	1180	294	332
	15 000	23 900	155	150	25.2	12.4	9.6	1010	1500	356	414
	20 000	31 400	169	165	27.5	13.7	10.4	1190	1770	408	486
	30 000	46 300	192	188	31.2	15.8	11.7	1490	2260	494	607
	50 000	75 500	226	222	32.3	19.0	13.6	1980	3050	630	804
	70 000	104 000	251	247	40.6	21.3	15.0	2390	3720	739	968
	100 000	146 000	281	277	45.3	24.2	16.7	2920	4600	875	1180
	150 000	216 000	320	316	51.4	27.9	18.8	3660	5850	1060	1470
	200 000	284 000	350	346	56.2	30.8	20.4	4300	6930	1210	1730
	300 000	418 000	398	395	63.7	35.5	23.0	5390	8810	1470	2160

Ro/ro ship	1000	2540	83	76	15.2	8.0	4.0	1210	1240	258	257
	2000	4820	107	99	18.1	10.9	5.2	1680	1700	348	357
	3000	7010	125	115	20.0	13.0	6.1	2020	2050	416	432
	5000	11 200	150	139	22.6	16.3	7.3	2560	2590	519	551
	7000	15 300	170	157	24.6	18.9	8.3	3000	3010	601	645
	10 000	21 300	194	179	26.8	22.1	9.5	3540	3550	702	764
	15 000	31 000	225	208	29.6	26.4	11.0	4270	4270	837	925
	20 000	40 400	250	231	31.8	30.0	12.3	4880	4860	949	1060
	30 000	58 800	290	269	35.1	35.8	14.3	5890	5850	1130	1280

Notes
* Full-load condition of wind lateral/front areas of log carrier do not include the areas of logs on deck
** Full-load condition of wind lateral/front areas of container ships include the areas of containers on deck

Type	Gross tonnage (t)	Displacement (t)	L_{OA} (m)	L_{BP} (m)	B (m)	D (m)	Max. draft (m)	Wind lateral area (m^2)		Wind front area (m^2)	
								Full load condition	Ballast condition	Full load condition	Ballast condition
Passenger ship	1000	1350	70	69	13.2	6.3	3.9	525	539	219	232
	2000	2500	90	86	15.7	8.2	5.1	842	855	295	310
	3000	3590	103	99	17.4	9.5	6.0	1110	1120	350	368
	5000	5650	123	117	19.8	11.6	7.3	1570	1570	435	456
	7000	7630	138	131	21.5	13.2	8.4	1970	1970	502	525
	10 000	10 500	156	147	23.5	15.1	8.7	2510	2500	585	609
	15 000	15 000	180	168	26.0	17.6	8.7	3310	3270	695	722
	20 000	19 400	199	185	28.0	19.7	8.7	4030	3960	785	815
	30 000	27 900	229	211	31.0	23.0	8.7	5310	5190	933	966
	50 000	44 000	273	250	35.2	27.9	8.7	7510	7300	1160	1200
	70 000	59 300	307	279	38.3	31.8	8.7	9440	9130	1340	1380
Ferry	1000	2240	79	72	17.0	6.9	4.9	449	466	175	179
	2000	4430	102	93	20.2	8.5	6.0	716	746	243	250
	3000	6590	118	108	22.3	9.6	6.7	941	982	295	305
	5000	10 900	142	131	25.4	11.3	7.8	1330	1390	376	390
	7000	15 100	160	148	27.6	12.5	8.7	1670	1750	441	459
	10 000	21 500	182	169	30.1	14.0	9.6	2120	2220	522	545
	15 000	31 900	210	196	33.4	15.8	10.8	2790	2930	632	664
	20 000	42 300	233	218	35.8	17.3	11.8	3380	3560	724	763
	30 000	63 000	270	253	39.6	19.6	13.3	4450	4690	877	928
	40 000	83 500	300	282	42.6	21.5	14.5	5400	5700	1010	1070

Gas carrier	1000	2910	75	70	12.5	6.2	5.3	457	511	151	166
	2000	5370	94	87	15.3	8.0	6.4	699	777	222	243
	3000	7680	106	99	17.3	9.2	7.2	896	992	278	303
	5000	12 000	124	116	20.1	11.1	8.4	1230	1350	369	401
	7000	16 200	138	129	22.2	12.5	9.2	1510	1660	444	481
	10 000	22 200	154	145	24.7	14.2	10.2	1870	2050	541	585
	15 000	31 700	174	165	27.9	16.4	11.5	2400	2620	677	730
	20 000	40 900	190	180	30.4	18.2	12.5	2870	3120	794	855
	30 000	58 500	216	205	34.2	21.0	13.1	3670	3990	994	1067
	50 000	91 800	253	241	39.9	25.2	13.1	5030	5430	1320	1410
	70 000	124 000	280	268	44.0	28.4	13.1	6180	6650	1590	1700
	100 000	169 000	313	300	49.0	32.3	13.1	7680	8250	1930	2060

20.4 Recommendation

As can be seen from the different tables in Section 20.2 the beam, the overall length and the draft may vary, depending on the country of origin of the ship and the ship's construction. The ship's dimensions may vary by as much as ±10 to ±15 per cent between the different sources. For important calculations and if the actual design ship's dimensions are not known, use of the information given in Section 20.3 from Japan could be recommended.

References and further reading

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21

Definitions

The following main definitions, as illustrated in Fig. 21.1, have been used in this book.

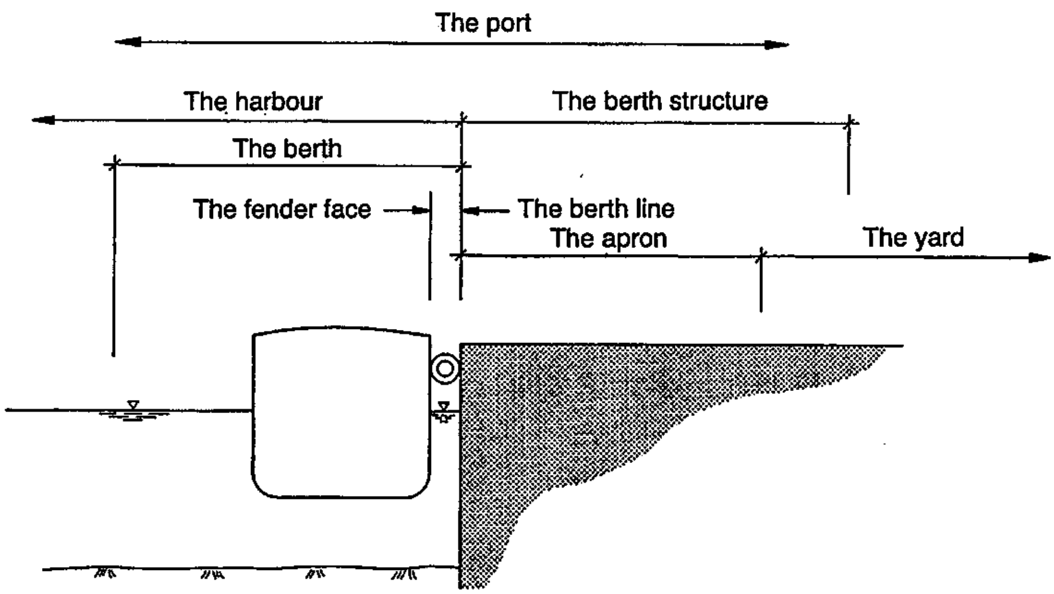

Fig. 21.1. Definitions.

There are currently numerous dictionaries and lexicons available covering almost all facets of the port, environment and navigation fields. In this book the following main terms and definitions, which are considered to be presently in common use in the field, especially by PIANC, have been used.

Accretion	Natural accretion — the build-up of land, solely by the action of the forces of nature, on a beach by deposition of water or airborne material.

Artificial accretion — similar build-up of land by reason of an act of man, such as the accretion formed by a groin, breakwater, or beach fill deposited by mechanical/hydraulic means.

Aerobic bacteria — Aerobic bacteria refers to the bacteria that oxidize substrate (feed) by oxygen respiration and live on the energy generated in the process. Aerobic bacteria are the opposite of anaerobic bacteria, which need no oxygen gas. Aerobic bacteria play an important role in the natural purification process of water bodies, the activated sludge process, the sprinkling filter method and other water quality preservation processes.

Air pollutant — Any substance in air, which could, if in high enough concentration, harm man, other animals, vegetation, or material. Pollutants may include almost any natural or artificial composition of matter capable of being airborne. They may be in the form of solid particles, liquid droplets, gases, or in combinations of these forms.

Generally, they fall into two main groups: (1) those emitted directly from identifiable sources; and (2) those produced in the air by interaction between two or more primary pollutants, or by reaction with normal atmospheric constituents, with or without photo activation. Exclusive of pollen, fog and dust, which are of natural origin, about 100 contaminants have been identified and fall into these categories: solids, sulphur compounds, volatile organic chemicals, nitrogen compounds, oxygen compounds, halogen compounds, radioactive compounds, and odours.

Anaerobic — Oxidation occurring in the absence of free or dissolved oxygen often facilitated by specific bacterial strains, e.g. methane-producing bacteria present during the anaerobic digestion of sewage sludge.

Anaerobic bacteria — Anaerobic bacteria refers to the bacteria that grow without dissolved oxygen. The decomposition of organic substances by anaerobic bacteria produces hydrogen sulphide, ammonia, methane and low molecular fatty acid.

	The process of digestion by anaerobic bacteria is applied to the treatment of human waste and sewage sludge.
Anchorage	That proportion of a harbour area or designated areas outside of the harbour in which ships are permitted to lie at anchor.
Apron	The area between the berth line and the transit shed or the storage area for loading and unloading of cargo.
Artesian water level	Hydrostatic pressure level higher than the ground level.
Astronomical tide	Tide due to gravitational attraction of the sun, moon and other astronomical bodies.
Ballast water	Water taken on board a vessel to ensure stability while navigating in an unladen or partially laden state.
Basin	Tidal — A dock or basin without water gates, in which the water level changes. (a) Turning — An area of water or enlargement of a channel used for turning around of ships. (b) Wet — An area of impounded water within which ships can remain afloat at a uniform level, independent of external tidal action.
Bathymetry	The physical configuration of the seabed, the measurement of depths of water in the ocean, etc., and also information derived from such measurement.
Belting	Substantially horizontal continuous narrow rigid steel fender along the ship side above the water line.
Berth	A place where the ship can moor. In the case of a quay or jetty structure it will include the section of the structure where labour, equipment and cargo move to and from the ship.
Berth line	A line along the outermost part of the superstructure. Removable equipment such as fenders will be on the outside of the berth line.
Berth structure	Artificial landing place for loading and unloading of ships. Berth structures can be subdivided into:

(a) Quay or wharf: a berth structure, which generally is aligned parallel to the shoreline.
(b) Jetty or pier: a berth structure which projects out into the water from the shore.
(c) Dolphin: a berth structure for mooring the ship on the open sea.

Best available techniques (BAT)	The latest stage in the development of activities, processes and their methods of operation which indicate the practical suitability of particular techniques as the basis of preventing or minimizing emissions to the environment.
Biochemical oxygen demand (BOD)	The amount of oxygen required during the aerobic decomposition of organic matter in a body of water. Or, a measure of the quantity of oxygen used in the biochemical oxidation of carbonaceous and nitrogenous compounds in a specified time, at a specified temperature and under specified conditions. The standard measurement is made for five days at 20 °C and is termed BOD_5. BOD is an indicator of the presence of organic matter in the water.
Biological treatment	Biological treatment refers to the method of treating waste water, sewage, human waste, etc., by utilizing the metabolism of organisms (bacteria, moulds, protozoa); it is mainly classified into aerobic treatment using aerobic organisms and anaerobic treatment using anaerobic organisms. *Aerobic treatment* involves decomposing organic substances in waste water into carbon dioxide, ammonia or water using aerobic organisms that are active when dissolved oxygen in water is sufficient. Aerobic treatment methods include the activated sludge process and sprinkling filtration. *Anaerobic treatment* involves decomposing organic substances in waste water into methane, carbon dioxide, hydrogen sulphide, ammonia or water, using anaerobic organisms that are active when dissolved oxygen in water is insufficient. Anaerobic treatment methods include methane fermentation, which is suitable for the treatment of high BOD industrial wastewater.

Bollard	A vertical post to which the eye of a mooring line can be attached.
Breakwater	A rubble mound and/or a concrete structure to protect the harbour area from wave action.
Breakwater berth	A berth structure on the leeside of the breakwater.
Breasting dolphin	A dolphin structure designed to take the impact from a berthing ship.
Bulkhead	A structure for retaining or to prevent earth or fill from sliding into water.
Bulkhead line	The farthest line offshore to which a fill or solid structure may be constructed.
Capital dredging	Dredging carried out to create new channels, etc., as distinct from maintenance dredging. This is also called new work dredging.
Channel	A dredged waterway through which ships proceed from the sea to the berth or from one berth to another within the harbour.
Chloride ion	Chloride ion refers to ionized chloride (Cl^-), which forms ionized compounds with various metals. Chloride ion is also found in natural water, measuring several ppm in river water and 1.9 per cent in seawater.
Chock	A guide for a mooring line, which enables the line to be passed through a ship's bulwark or other barrier.
Coastal berth	Berth fully exposure to wind, wave and current.
Confined dredged material placement	Placement of dredged material within disked near shore or upland confined placement facilities that enclose and isolate the dredged material from adjacent waters during placement. Confined dredged material placement does not refer to sub-aqueous capping or contained aquatic dredged material placement.
Containment (retention) basin	Containment constructed of dikes for the purpose of retaining dredge spoil until much of the suspended material has settled out when the water itself is released.

Contaminated sediment or contaminated dredged material	Sediment or material that has an unacceptable level of contaminant(s), which have been demonstrated to cause an unacceptable adverse effect on human health or the environment.
Convention on the Prevention of Marine Pollution by the Dumping of Wastes and Other Matter	This Convention was prepared at the United Nations Conference on the Human Environment (Stockholm) in June, 1972, adopted on 13 November in the same year in London and took effect on 30 August, 1975. The Convention mainly contained (1) an absolute ban on the ocean dumping of organic halogen compounds and mercury; (2) the issuance of a special license for the ocean dumping of lead, copper and zinc; and (3) the issuance of a general license for the ocean dumping of substances other than the above. Upon the issuance of either special or general licenses, the shape and characteristics of substances to be dumped; the location, water depth, and distance from land of dumping sites, and pre-dumping treatment methods must be taken into consideration in establishing criteria.
Debris	Wastes or remains of something broken down or destroyed. Also, any oversized material adversely affecting the hydraulic transport system.
Density	The total weight of all materials/unit of volume.
Deposit (geology)	Deposit refers to the matter made of crushed bits of disintegrated rocks, carcasses, volcanic ejecta, etc. and physically and chemically settled and accumulated in water or on the land. In other words, it is clay, sand or gravel formed by weathering or erosion and transported by rivers or sea currents to sink and settle on the bottom of the sea or lake.
Design life	The period of time that goes from the beginning of the construction of the structure until it is dismantled, put out of service or used for another purpose.
Diffraction of water waves	When a part of a train of waves is interrupted by a barrier e.g. breakwater, the effect of diffraction is manifested by propagation of waves into sheltered region within the barrier's geometric shadow.

Dock	A harbour basin where the basin is cut off from the tides by dock gates.
Dolphin	An isolated piled or gravity structure used either to manoeuvre a ship or to facilitate holding it in position at its berth.
Dredged material	Material excavated from inland or ocean waters. The term dredged material refers to material which has been dredged usually from the bed of a water body, while the term sediment refers to material in a water body prior to the dredging process.
Dredging	Dredging refers to loosening and lifting earth and sand from the bottom of water bodies. Dredging is often carried out to widen the stream of a river, deepen a harbour or navigational channel, or collect earth and sand for landfill; it is also carried out to remove contaminated bottom deposit or sludge to improve water quality.
Dry density	The total weight of solids only/unit of volume. (Also see: density.)
Ecosystem	Ecosystem refers to the system of life formation by organisms (animals, plants, etc.) and close interactions between them and their physical environment. Or, a natural unit consisting of living and non-living parts interacting with each other, formed by the organisms of a natural community and their environment.
Environmental assessment (EA)	A written environmental analysis, which is conducted to determine whether a proposed undertaking would significantly affect the environment. The conducting of an environmental assessment for a proposed project is usually a mandatory requirement of various jurisdictional authorities. (Also see environmental impact assessment, EIA.)
Environmental impact assessment (EIA)	Environmental impact assessment refers to a system involving the investigation, estimation and evaluation of the effect of a project or activity on the environment and is usually conducted by the proponent for the proposed undertaking in the process of planning. (Also see environmental assessment, EA.)

Environmental management plan	Environmental management plans are required to reflect socio-economic trends and the general public needs, with air, water, forests, soil and other natural elements viewed as a whole, to preserve and utilize these resources appropriately to prevent destruction and to create a comfortable environment. Governments play an important role in promoting such environmental policies. A regional environmental management plan conducted by a local government aims at identifying what an ideal regional environment is in consideration of regional nature and social conditions and community intentions. It also aims at integrating pollution control, environmental preservation and improvement, and implementing relevant activities according to well-planned procedures and schedules. Regional environmental planning aims at developing conditions necessary for the ideal environment, through mediation for organizations and communities in regard to environmental preservation and utilization from a long-term perspective. A plan that describes specific conservation and protection actions that will be undertaken during project planning, construction, operation, and maintenance to lessen the effects of the project on the environment and to ensure that sustainable development is achieved; it includes real time and retroactive monitoring of project effects.
Erosion	A natural physical process where wind, wave, rain or surface water run-off loosen and remove soil particles from land surfaces often deposited in rivers and lakes.
Estuarial berth	More wave exposure than tidal basins and with maximum tidal range and current.
Fairway	Open water of navigable depth.
Gravity wall	A retaining wall of heavy cross-section that resists the horizontal loads by its own deadweight.
Groin	A shore protection structure. Usually built perpendicular to a coast line to retard littoral transport of sedimentary materials.

Groundwater	All subsurface water that fills voids between highly permeable ground strata comprised of sand, gravel, broken rocks, porous rocks, etc. and moves under the influence of gravitation. On the Earth, besides the seawater and the Antarctic water, far more groundwater exists than water in rivers, lakes and marshes.
Harbour	Protected water area to provide safe and suitable accommodation for ships for transfer of cargo, refuelling, repairs, etc. Harbours may be subdivided into: (a) Natural harbours: harbours protected from storms and waves by the natural configuration of the land. (b) Semi-natural harbours: harbours with both natural and artificial protection. (c) Artificial harbours: harbours protected from the effect of waves by means of breakwaters, or harbours created by dredging.
Hardness	The hardness of water is indicated by the content of dissolved calcium and magnesium salts. Calcium and magnesium salts that are transformed to insoluble salts by boiling denote temporary hardness, while calcium and magnesium salts that do not settle when boiled denote permanent hardness. The sum of temporary and permanent hardness is called total hardness.
Hawser	A synthetic or natural fibre rope or wire rope used for mooring or towing.
Hydrocarbons	Compounds found in fossil fuels that contain carbon and hydrogen and may be carcinogenic.
Hydrography	The description and study of seas, lakes, etc.
Impounded basins	Lock basins with approximately constant water level and no waves and current.
Jetty	A berth structure which projects out into the water from the shore, or a berth structure at some distance from the shoreline.
Jetty head	A platform at the seaward side of a jetty.
Lo/Lo	Lift on/lift off vessels that are loaded and discharged through the hatchways.

Marginal berth	A berth structure parallel to the shore.
Marine pollution	Among international efforts to control marine pollution, the MARPOL 73/78 Convention governs pollution by ships while the Convention on the Prevention of Marine Pollution by the Dumping of Wastes and Other Matter (Dumping Convention) governs the ocean dumping and incineration of waste materials.
Messenger line	A light line attached to the end of a main mooring line and used to assist in heaving the mooring to the shore or to another shil.
Mineral oils	Residues of natural fossil fuels.
Mooring dolphin	A dolphin structure equipped with bollards or quick-release hooks for mooring the ship.
Navigable waters	Traditionally, waters sufficiently deep and wide for navigation by all, or specified sizes of vessels.
Oil boom	Oil boom is a device comprised of a large float with a suspended screen attached underneath. It is used to contain spilled oil from spreading further on the sea surface or to protect a given sea area, such as a fish farm, from pollution.
Oil separator	Oil separator is a device that isolates oil from water by flotation treatment.
Organic materials	Compounds composed of carbon, hydrogen and other elements with chain or ring structures. Almost all chemical constituents of living matter are of this type, but very many compounds of this type are manufactured and do not occur naturally.
Parts per million	This is a weight/volume or weight/weight measurement used in contaminant analysis. It is interchangeable with 'milligrams/l' or 'milligrams/kg' in the case of liquids. Chemical dosages are often referred to as parts/million, e.g., 100 ppm of polymer. 100 ppm = 0.01 per cent.

pH — A measure of the acidity or basicity of a solution, i.e. the negative of the logarithm of the hydrogen ion concentration; 'Hydrogen ion exponent', a unit for measuring hydrogen ion concentrations. A scale (0–14) represents an aqueous solution's acidity or alkalinity. Low pH values indicate acidity and high values, alkalinity. The scale's mid-point, 7, is neutral. Substances in an aqueous solution ionize to various extents, giving different concentrations of H and OH ions. Strong acids have excess H ions and a pH of 1–3 (HCl, pH = 1). Strong bases have excess OH ions and a pH of 11–13 (NaOH, pH = 12).

Pier — A berth structure projecting out from the shore line.

Port — A sheltered place where the ship may receive or discharge cargo. It includes the harbour with its approach channels and anchorage places. The port may be subdivided into:
(a) Ocean ports: ports located on coasts, tidal estuaries or river mouths where the port can be reached directly by ocean going ships.
(b) Inland waterway ports: ports located on navigable rivers, channels and lakes.

Port side — Left side of the ship looking towards the bow.

ppb — Parts per billion, commonly considered equivalent to micrograms/l or kilogram (Φg/l or Φg/kg).

ppm — Parts per million, commonly considered equivalent to milligrams/l or kilogram (mg/l or mg/kg).

ppt — Parts per trillion, commonly considered equivalent to nanograms/l or kilogram (ng/l or ng/kg).

Precipitation — Rainfall, snowfall or any condensate.

Quay — A berth structure parallel to the shore line.

Refraction of water waves — The process by which the direction of incoming waves in shallow water is altered due to the contours of the seabed.

Relieving platform A platform or deck structure built below the top deck level and supported on bearing piles. The main function of the platform is to reduce the lateral soil pressures over the upper portion of the sheet wall.

Risk assessment In dealing with environmental problems, a certain degree of uncertainty is unavoidable, despite advances made thus far in the scientific elucidation of negative impact (or risk) on human beings and the natural environment. However, irreversible damage could be made if necessary measures were delayed until complete scientific elucidation is achieved.

In such a situation, an integrated policy-making approach of two processes, scientifically estimating and evaluating the negative impact of human activities on humans and the environment (risk assessment) and deciding and executing rational policies for risk mitigation based on risk assessment (risk management), is becoming established. International agreement made on the protection of the ozone layer is a precedent of this approach.

Riprap A wall or foundation made of broken stones thrown together irregularly or loosely in water or on a sea bottom.

Ro/ro Roll on/roll off ships that are loaded and discharged by way of ramps.

Sea island A berth structure with no direct connection to the shore, at which the ships can berth. Berthing can take place on either one or both sides of the structure.

Sediment Materials such as sand, silt or clay suspended in or settled on the sea bottom. Solid fragmental material originating from weathering or rocks or by other processes and transported or deposited by air, water or ice, or that accumulated by other processes such as chemical precipitation from solution or secretion by organisms. The term is usually applied to material held in suspension in water or recently deposited from suspension and to all kinds of deposits, essentially of unconsolidated materials.

Sheet wall A retaining wall that resists loading.

Silt	Fine particulate organic and inorganic material; strictly confined to material with an average particle size intermediate between those of sands and clays, but often taken to include all material finer than sands.
Silt curtain	A curtain or screen suspended in the water to prevent silt from escaping from an aquatic construction site.
Starboard side	Right side of the ship looking towards the bow.
Tail	A short length of synthetic rope attached to the end of a wire mooring line to provide increased elasticity and also ease of handling.
Tidal basins	Greater range of water levels.
Terms of Reference (TORs)	A statement of the specific work to be done under a consulting agreement or similar contract.
TEU	20 ft container, twenty equivalent units.
Toxicity	The degree of danger posed by a substance to animal or plant life. Level of mortality by a group of organisms that have been affected by the properties of a substance, such as contaminated water, sediment, or dredged material. A term describing the limit of intolerance of organisms to survive lethal chronic or short-term subjection to certain chemical and contaminating substances, or physical and environmental conditions.
ULCC	Ultra large crude carrier used for ship with deadweight greater than 400 000 dwt.
VLCC	Very large crude carrier with deadweight between 140 000 dwt and 400 000 dwt.
Yard	The yard is subdivided into: (a) The primary yard, which is the section of the yard adjacent to the apron, and primarily used for temporary storage of inbound and outbound cargo (the storage area). (b) The secondary yard, which is the section of the yard used for chassis and empty container storage, miscellaneous equipment and facilities.

References and further reading

British Standard BS 6349 (2000) *Maritime Structures. Part 1: Code of Practice for General Criteria*, London: BSI.

The Permanent International Association of Navigation Congresses (PIANC) established an International Permanent Environment Commission (PEC) in 1992.

US Army Coastal Engineering Research Center (1984) *Shore Protection Manual. Volumes 1 and 2* (1984) Department of the Army Corps of Engineers.

22
Conversion factors

In this book the metric units are used. When the word 'ton' (tonne) is used. this is a metric ton.

For conversion between the imperial or the US system and the metric system the following conversion factors should be used:

22.1 Length

1 mm	= 0.03937 in	1 in	= 25.4 mm	
1 m	= 3.281 ft	1 ft	= 12 in	= 0.3048 m
	= 1.094 yd	1 yd	= 3 ft	= 0.9144 m
1 km	= 0.6214 mile	1 mile	= 1760 yd	= 1.609 km

1 fathom	= 6 ft	
1 cablelength (UK)	= 100 fathoms	= $\frac{1}{10}$ nautical mile
	= 185.2 m	
1 nautical mile	= 10 cablelengths = 6080 ft = 1852 m	
	$\frac{1}{60}$ of a meridian degree	
1 degree	= 60 nautical miles	

As a curiosity:

1 Swedish inch	= 2.47 cm
1 English inch	= 2.54 cm
1 Russian inch	= 2.54 cm
1 Norwegian inch	= 2.62 cm
1 Danish inch	= 2.67 cm
1 Paris inch	= 2.71 cm

22.2 Speed

1 km/h = 0.278 m/sec = 0.62 mph = 0.54 knots

3.60 km/h	= 1 m/sec	= 2.24 mph	= 1.94 knots
1.61 km/h	= 0.45 m/sec	= 1 mph	= 0.87 knots
1.85 km/h	= 0.5145 m/sec	= 1.15 mph	= 1 knot

22.3 Area

1 mm^2	= 0.00155 in^2	1 in^2	= 645.2 mm^2
1 m^2	= 10.76 ft^2	1 ft^2	= 0.0929 m^2
	= 1.196 yd^2	1 yd^2	= 0.8361 m^2
1 ha	= 10000 m^2		
	= 2.471 acres	1 acre	= 0.4047 ha = 4047 m^2
1 km^2	= 0.3861 $mile^2$	1 $mile^2$	= 2.59 km^2

22.4 Volume

1 m^3	= 1000 litres	= 35.3147 ft^3 = 1.308 yd^3
1 ft^3	= 0.02832 m^3	
1 yd^3	= 0.7646 m^3	
1 litre	= 0.22 imperial gallons	= 0.2642 US gallons
4.546 litres	= 1 imperial gallon	= 1.201 US gallons
3.785 litres	= 0.8327 imperial gallons	= 1 US gallon
1 pint (UK)		= 0.5683 litres
1 pint (US)		= 0.4732 litres
1 US barrel	= 5.6146 ft^3	= 158.99 litres

22.5 Weight

1 kg	= 1000 g	= 2.2046 lb	
1 tonne	= 1000 kg	= 2204.6 lb	
	= 0.98421 long tonne		
	= 1.10231 short tonne		
1 hundredweight (1 cwt)	= 112 lb	= 50.802 kg	
1 lb	= 0.4536 kg		
1 long ton	= 2240 lb	= 20 cwt	= 1016 kg
1 short ton	= 2000 lb	= 907.3 kg	
1 kip (US)	= 1000 lb	= 453.6 kg	

22.6 Force

1 N	= 0.2248 lb	= 0.1020 kg

4.448 N	= 1 lb	= 0.4536 kg	
9.807 N	= 2.205 lb	= 1 kg	= 1 daN
1 kN	= 0.1004 long tonne	= 102.0 kg	= 0.1020 tonne
9.964 kN	= 1 long tonne	= 1016 kg	= 1.016 tonne
9.807 kN	= 0.9842 long tonne	= 1000 kg	= 1 tonne

22.7 Force per unit length

1 N/m	= 0.06852 lb/ft	= 0.1020 kg/m
14.59 N/m	= 1 lb/ft	= 1.488 kg/m
9.807 N/m	= 0.672 lb/ft	= 1 kg/m
1 kN/m	= 0.0306 long tonne/ft	= 0.1020 tonne/m
32.69 kN/m	= 1 long tonne/ft	= 3.333 tonne/m
9.807 kN/m	= 0.3000 long tonne/ft	= 1 tonne/m

22.8 Force per unit area

1 N/mm^2	= 145.0 lb/in^2	= 10.20 kg/cm^2
0.006895 N/mm^2	= 1 lb/in^2	= 0.0703 kg/cm^2
0.09807 N/mm^2	= 14.22 lb/in^2	= 1 kg/cm^2 = 10 tonne/m^2
1 N/m^2	= 0.02089 lb/ft^2	= 0.102 kg/m^2
47.88 N/m^2	= 1 lb/ft^2	= 4.882 kg/m^2
9.807 N/m^2	= 0.2048 lb/ft^2	= 1 kg/m^2
10 tonne/m^2	= 1 kg/cm^2	= 0.1 MPa = 0.1 N/mm^2 = 100 kN/m^2
1 atmosphere	= 1 kg/cm^2	= 10 m water pressure

22.9 Moment

1 Nm	= 8.851 lbin	= 0.7376 lbft	= 0.1020 kgm
0.1130 Nm	= 1 lbin	= 0.08333 lbft	= 0.01152 kgm
1.356 Nm	= 12 lbin	= 1 lbft	= 0.1383 kgm
9.807 Nm	= 86.80 lbin	= 7.233 lbft	= 1 kgm
1 tonne m	= 3.229 long tonne ft		
1 long tonne ft	= 0.3097 tonne m		

22.10 Temperatures

Celsius (C°) = (F° − 32)/1.8 = (F° − 32)5/9

Fahrenheit (F°) = 1.8 C° + 32

22.11 Useful data

Standard gravitational acceleration	$= 9.807\ m/sec^2$	$= 32.174\ ft/sec^2$
Density of water	$= 1000\ kg/m^3$	$= 62.4\ lb/ft^3$
Weight of reinforces concrete	$= 23.6\ kN/m^3$	$= 2400\ kg/m^3$
		$= 150\ lb/ft^3$
1 horsepower (HP)	$= 0.746$ watts	

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Note: Page number in *italics* refer to illustrations, tables or figures.

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PORT DESIGNER'S HANDBOOK: RECOMMENDATIONS AND GUIDELINES

Over the past twenty years or so, the growing interest in the design of port and harbour structures has produced a huge amount of research, technical papers and information, especially from the International Navigation Association PIANC, many of which have been evaluated and summarized in this title. *Port designer's handbook: recommendations and guidelines* looks at the main issues and assumptions in the layout, design and construction of modern port structures, and the forces and loadings acting upon them.

Based on 40 years of experience, *Port designer's handbook: recommendations and guidelines* supercedes *Port Design: guidelines and recommendations*, written by the author and published 1988. It reflects the latest progress and development in navigation safety, port planning and site selection, layout of container and oil and gas terminals, cargo handling, berth design and construction, use of concrete for berth structures in the marine environment, deterioration and methods of repair and fender and mooring principles. The title also covers evaluation of the operational conditions due to the impact from wind, waves and current loads on port and berth structures, and berthing and layout criteria for ships in channels and harbour basins. Equally relevant is the evaluation of the different practices for design and construction of port and berth structures, and on recommendations given by the different international harbour standards and recommendations.

Some of the key topics discussed include:

- Port planning and site selection
- Channels and harbour basins
- Environmental forces, operational conditions and acceptable ship movements
- Manoeuvring and berthing requirements
- Safety considerations in layout and design of berth structures
- Design loads on berth structures and impacts from ships
- Criteria for choice of berth structures
- Solid and open berth structures
- Container terminals
- Fender design and berth details
- Erosion protection
- Corrosion of steel sheet piles
- Different methods of underwater concreting
- Deterioration and repair of concrete in marine environments
- Ship dimensions and ship wind and current areas for different types of ship

Port designer's handbook: recommendations and guidelines will be invaluable for the practising port and harbour engineer who is involved in the layout, design and construction of berth and harbour structures.

Carl A. Thoresen

During his professional career with the largest Norwegian consulting company, Norconsult A.S, Carl has been chief port and harbour engineer and consultant for more than 500 different port and harbour projects around the world.

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He is author of *Port Design: Guidelines and Recommendations* (1988) co-author of the PIANC publications *Development of Modern Marine Terminals* (1986) and *Fender System 2002* (2002), and co-author of *Norwegian Recommendations for Waterfront Structures* (1988) and *Norwegian Recommendation for Concrete Construction in Water* (latest edition 2003). He has written about 50 papers and articles on port and harbour planning, berth and fender design and constructions and marine repair and rehabilitation works.

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