Third Edition

Design of Marine Facilities

ENGINEERING FOR PORT AND HARBOR STRUCTURES

John W. Gaythwaite, P.E., D.PE, D.CE

ASCE PRESS

Design of Marine Facilities

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On the cover
A modern container ship berthed in Hamburg, Germany.
Photo courtesy of Chriss73/iStockPhoto.

To My Wife,
Michele Rabot Gaythwaite

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Contents

Foreword

Engineers specializing in port/harbor structures and ocean engineering know that there is no "code" providing specific guidance for the analysis, design, and rehabilitation of marine facilities. This third edition of *Design of Marine Facilities* is a comprehensive treatise on the subject, including such varied topics as vessel characteristics, design and environmental considerations, all possible environmental loads, mooring and berthing analysis and design, fixed structural design, geotechnical considerations, dry docks, and inspection, deterioration, and rehabilitation.

Having known John Gaythwaite for more than 10 years, I can attest to his 40-plus years of experience in this field. His specific background makes him eminently qualified to write the text that guides marine engineers around the world. In 2010, he received the prestigious ASCE/COPRI Moffatt and Nichol Harbor and Coastal Engineering Award for his continued contributions in this field and his dedication to the civil engineering profession. He has been a long-time member of the Permanent International Association of Navigation Congresses (PIANC), the Society of Naval Architects and Marine Engineers (SNAME), and the American Society of Civil Engineers (ASCE). In addition, Mr. Gaythwaite is an experienced sailor and holds a U.S. Coast Guard master's license for vessels up to 100 tons.

In this new third edition, many of the chapters have been updated and expanded. Some of the new sections include extensive guidance on seismic loads and analysis, inspection and rehabilitation, and more unusual loads, such as the loads of a passing vessel onto moored vessels. Climate change and sea level rise are also covered. The list of references has been increased to provide additional in-depth explanations and guidance. The latest codes and recommended practice are incorporated, along with some new ASCE/COPRI standards and recommended practices. The appendixes provide websites, internet sources, and the latest conferences and periodicals.

To the practicing engineer or student interested in this exciting field of marine structural engineering, this text is an invaluable reference and guidance document. And as I continue to teach this subject at the graduate level, this edition will be my text choice, because it stands out as the best available guidance and reference document for port and harbor structures in the United States.

Martin L. Eskijian
April 2016

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Preface

A lot has happened in the field of port engineering and marine facilities design in a little more than a decade since the second edition, and almost three decades since the first edition, of this book was published. These changes include the proliferation of information and availability of data via electronic media; advances in materials and construction methods and capabilities; advances in knowledge and understanding of environmental forces; sophistication of computational techniques; added constraints of port security, sustainability, environmental, and regulatory issues; and the development of a global economy, with its effects on shipping, ship size, and so on. This book was originally written to fill a niche in the general literature on port and harbor engineering, and it is intended to provide the practicing civil engineer with essential background information to understand the design of port and harbor structures that make up various marine facilities for the berthing, mooring, and repair of vessels. It provides an introduction and overview of the subject that should also prove useful to port authority engineers, marine terminal operators, marine contractors, port planners, and others with an interest in understanding the basics of marine facilities design. The book is comprehensive enough to serve also as a text for an introductory course in port and harbor structure design. This third edition has been thoroughly revised and updated throughout, with expanded coverage, including new material on seismic design, tsunamis in ports, and other topics.

This book also attempts to serve as a link between research and design practice. An in-depth treatment of all the subject matter of this book would increase its size many times, so frequent references are made to sources that provide further design guidance and development of the theoretical background. As there are no strict step-by-step design codes or standards in the field, the marine structures engineer must be familiar with a wide range of information sources and must be prepared to consult the literature relevant to a particular project, which often engages the international community. Accordingly, important international standards and sources are cited throughout.

Organizing a book that draws from various disciplines and covers a broad area of study is a difficult undertaking, as one must provide explanations or definitions that in turn depend upon subsequent discussions. Therefore, there is considerable cross-referencing between chapter sections that is intended to help the reader grasp the interrelationships of all the subject matter. The book is organized to flow somewhat as the design process does: from a description of design vessels; to the establishment of general design criteria; to the evaluation of loadings; to structural design; and to functional design considerations, including rehabilitation, maintenance, repair, and inspection; assessment; and asset management of existing facilities.

The construction of important port and harbor works is often a major civil engineering undertaking, requiring the specialized expertise and concerted efforts of many experienced individuals. Likewise, this book owes its genesis to the many contributors to the large and growing body of literature on the subject, such as those cited in the references. I am particularly indebted to the many individuals who have contributed materials and comments directly to this book, including those who contributed to the previous editions as well. I gratefully acknowledge the contributing authors to this edition: David R. Carchedi, Ph.D., P.E., and Russell J. Morgan, P.E., who have revised and updated the chapter on geotechnical considerations that they originally authored in the first edition of this book. They were assisted by Joann Marseglia, who provided valuable production support. Robert Heger, P.E.; Mark Procter, P.E.; and Douglas Pearlson have thoroughly updated the chapter on dry docks originally authored by Paul M. Becht, P.E., and James R. Hetherman, P.E. The staff of Appledore Marine Engineering, LLC, under the guidance of Noah J. Elwood, P.E., president, have contributed an entirely new chapter on inspection and asset management, as well as providing support and materials in other chapters. A special note of thanks goes to David L. Marcotte, P.E., for his assistance with many of the book's figures and final artwork and the appended collection of Internet sources. Special thanks to Martin L. Eskijian, P.E., D.PE, for his review and valuable comments on the new sections on seismic design and tsunamis in ports and other valuable input. Thanks to Omar Jaradat, Ph.D., P.E., for his review of the section on seismic loads. Thanks to Ronald Byres, P.E., P.Eng., for providing certain photos. Finally, a note of thanks to the peer reviewers, whose efforts and valuable review comments served to improve the final text.

John W. Gaythwaite
Manchester, MA

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Notations

The following symbols and abbreviations occur most often in the text. Notations that are not repeated in the text are defined where introduced and may not appear below. Certain characters are used frequently with different subscripts, which are defined where first introduced and are denoted by an asterisk below.

A^*	Area, as defined in text
a	Acceleration
a^*	Motion amplitude, as defined in text
B	Vessel beam or width
b	Net buoyancy force
bhp	Brake horsepower
BWL	Vessel beam at waterline
C	Wave celerity
C^*	Coefficient, as defined in text
c.b.	Center of buoyancy
CEU	Car equivalent unit
c.f.	Cubic feet
c.g.	Center of gravity
c.m.	Cubic meters
D	Vessel draft: pile diameter or width normal to flow
D^*	Diameter or dimension as defined in text
D'	Vessel projected width normal to flow
$\bar{D}$	Characteristic pile embedment length
d	Water depth
DT	Displacement tonnage
DWL	Design water level or design waterline length of vessel
DWT	Deadweight tonnage
E^*	Energy or encounter probability, as defined in text
e	Exponential (base of Naperian logarithms)
F^*	Force, as defined in text
f	Frequency = $1/T$
f^*	Compressive or bending strength, as defined in text
FB	Vessel or floating structure freeboard
fps	Feet per second
g	Acceleration of gravity (usually taken as 32.16 ft/s^2)

gal./min	Gallons per minute
GRT	Gross registered tonnage
H^*	Wave height, as defined in text
h^*	Height or vertical distance, as defined in text
HAT	Highest astronomical tide
K^*	Force factor, spring constant, or coefficient, as defined in text
k	Wave number = $2\pi/L$
k^*	Constant, as defined in text
L^*	Wavelength or structure length, as defined in text
l^*	Length of line, cable, or pile, as defined in text
LAT	Lowest astronomical tide
lb/ft	Pounds per linear foot
lb/ft^2	Pounds per square foot
lb/ft^3	Pounds per cubic foot
LBP	Vessel length between perpendiculars
LOA	Vessel length overall
l.t.	Long ton (= 2,240 lb)
LWL	Vessel length on waterline
LWT	Vessel light displacement (tonnage)
M^*	Mass or moment, as defined in text
M'	Virtual mass
m^*	Mass factor, as defined in text
MHW	Mean high water
MHHW	Mean higher high water
MLLW	Mean lower low water
MLW	Mean low water
MSL	Mean sea level
m.t.	Metric ton (= 2,205 lb), often denoted as "tonne"
MTI	Moment to trim 1 in.
MTL	Mean tide level
N^*	Nondimensional scale number, as defined in text
n	Number or quantity
NAVD	North American vertical datum
NGVD	National geodetic vertical datum (≡ MSL of 1929)
n_h	Soil subgrade modulus
NRT	Net registration tonnage
P	Pressure, force per unit area, or member axial load, as defined in text
$P_{(*)}$	Probability of occurrence of *, as defined in text
pH	Negative log of hydrogen ion concentration (measure of acidity)
PPI	Pounds per inch immersion
Q	Volume flow rate
R^*	Reaction force, as defined in text
r	Radius or dimension, as defined in text
RAO	Response amplitude operator
rms	Root mean square value
R_T	Return period

S	Dimension, as defined in text
s	Length along curve or catenary
S_c	Anchor line scope
shp	Shaft horsepower
SWL	Still water level
$S(\omega)$	Wave height spectral density
T^*	Wave period or structure period, as defined in text
t	Time or duration
TEU	Twenty-foot equivalent unit (vessel containers)
U^*	Current or water particle velocity, as defined in text
u^*	Velocity component, as defined in text
V^*	Wind or vessel velocity, as defined in text
v^*	Velocity component, as defined in text
W^*	Weight, as defined in text
w^*	Velocity component or weight per unit length, as defined in text
x,y,z	Coordinate directions or distance along x,y,z direction
α^*	Angle or angular acceleration, as defined in text
β	Characteristic length of elastic beam or angle, as defined in text
γ	Unit weight or specific weight
Δ	Vessel displacement in weight units (inverted ∇ indicates vessel displacement in unit volume)
δ^*	Displacement or deflection, as defined in text
ε	Horizontal excursion of moored vessel or structure
η_c	Wave crest elevation above SWL
θ	Angle, as defined in text
μ	Coefficient of fiction
ν	Kinematic viscosity
ρ	Mass density $(=\gamma/g)$
σ^*	Soil stress, as defined in text
τ	Shear or torsional stress, as defined in text
ϕ	Angle, as defined in text
ψ	Angle, as defined in text
ω	Circular frequency = $2\pi/T$ or angular acceleration

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Nomenclature

AA	Aluminum Association
AAPA	American Association of Port Authorities
AASHTO	American Association of State Highway and Transportation Officials
ABS	American Bureau of Shipping
ACI	American Concrete Institute
ACOPNE	Academy of Coastal, Ocean, Port and Navigation Engineers (ASCE)
ADA	Americans with Disabilities Act
AFPA	American Forest & Paper Association
AGA	American Galvanizers Association
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
AITC	American Institute of Timber Construction
AMS	American Meteorological Society
ANSI	American National Standards Institute
API	American Petroleum Institute
AREMA	American Railway Engineering and Maintenance-of-Way Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
AWC	American Wood Council
AWPA	American Wood Protection Association
AWPI	American Wood Preservers Institute
AWS	American Welding Society
BSI	British Standards Institute
CEM	*Coastal Engineering Manual* (USACE)
CERC	Coastal Engineering Research Center (USACE)
CFR	Code of Federal Regulations
CHL	Coastal and Hydraulics Laboratory (USACE)
CI	Cordage Institute
CIRIA	Construction Industry Research and Information Association (UK)
COPRI	Coasts, Oceans, Ports, and Rivers Institute (ASCE)
CRREL	Cold Regions Research and Engineering Laboratory (USACE)
CRSI	Concrete Reinforcing Steel Institute
CSI	Construction Specifications Institute
DFI	Deep Foundations Institute

DHI Danish Hydraulic Institute
DHS Department of Homeland Security
DNV Det Norske Veritas (Norwegian Classification Society)
DOD Department of Defense
EAU (German Society for Harbor, Soils and Foundation Engineering)
EM *Engineering Manual* (USACE)
EPA Environmental Protection Agency
ERDC Engineering Research and Development Center (USACE)
FEMA Federal Emergency Management Agency
FHWA Federal Highway Administration
FMC Federal Maritime Commission
IADC International Association of Drilling Contractors
IAHR International Association for Hydro-Environmental Engineering and Research
IALA International Association of Lighthouse Authorities
IAPH International Association of Ports and Harbors
IBC International Building Code
ICC International Code Council
ICHCA International Cargo Handling Coordination Association
ICRI International Concrete Repair Institute
IMCO Intergovernmental Maritime Consultative Organization
IMO Intergovernmental Maritime Organization
INA International Navigation Association (PIANC)
IPCC Intergovernmental Panel on Climate Change
ISO International Standards Organization
JSE *Journal of Structural Engineering* (ASCE)
JWPCOE *Journal of Waterway, Port, Coastal, and Ocean Engineering* (ASCE)
MARAD Maritime Administration (U.S.)
MarCom Maritime Commission (PIANC)
MARIN Maritime Research Institute of the Netherlands
MOTEMS Marine Oil Terminals Engineering and Maintenance Standards
NACE National Association of Corrosion Engineers
NAVFAC Naval Facilities Engineering Command
NAVSEA Naval Sea Systems Command
NCDC National Climatic Data Center
NCEL Naval Civil Engineering Laboratory
NCHRP National Cooperative Highway Research Program
NDBC National Data Buoy Center
NDS National Design Specification (AWC)
NEHRP National Earthquake Hazard Reduction Program
NFESC Naval Facilities Engineering Service Center
NFPA National Fire Protection Association
NGS National Geodetic Survey
NIST National Institute of Standards and Technology
NOAA National Oceanic and Atmospheric Administration
NOS National Ocean Service

NRC	National Research Council
NWS	National Weather Service
OCADI	Overseas Coastal Area Development Institute (Japan)
OCIMF	Oil Companies International Marine Forum
OECD	Organisation for Economic Co-operation and Development
OSHA	Occupational Safety and Health Administration
OTC	Offshore Technology Conference
PCA	Portland Cement Association
PCI	Precast/Prestressed Concrete Institute
PIANC	Permanent International Association of Navigation Congresses
POLA	Port of Los Angeles
POLB	Port of Long Beach
RINA	Royal Institute of Naval Architects
ROM	Recommendations for Maritime Works (Spain)
SIGTTO	Society of International Gas Tankers and Terminal Operators
SNAME	Society of Naval Architects and Marine Engineers
SPC	Southern Pine Council
SPIB	Southern Pine Inspection Bureau
SSPC	Steel Structures Painting Council (Society for Protective Coatings)
TCLEE	Technical Council on Lifeline and Earthquake Engineering
UFC	Unified Facilities Criteria
UNCTAD	United Nations Conference on Trade and Development
USACE	U.S. Army Corps of Engineers
USCG	U.S. Coast Guard
USDOT	U.S. Department of Transportation
USGS	U.S. Geological Survey
USSI	U.S. Standards Institute
WBDG	Whole Building Design Guide
WCLIB	West Coast Lumber Inspection Bureau
WES	Waterways Experiment Station (USACE)
WIS	Wave Information Studies (USACE)
WMO	World Meteorological Organization
WODA	World Organisation of Dredging Associations
WRI	Wire Reinforcement Institute
WRTB	Wire Rope Technical Board

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CHAPTER 1

Introduction

The construction of port and harbor works was among the earliest major undertakings of civilized human beings. The ancients often displayed a great intuitive understanding of nature in their port works, which, unfortunately, have been lost in the decline of empires and changing coastlines. The timber and stone harbor works of less than 100 years ago have been mostly replaced with concrete and steel structures that have extended offshore port facilities into deeper water at exposed locations. Even so, port and harbor engineers depend heavily upon the collection of past experiences to temper their analysis and contemporary design practices.

This book, which reflects the rapid progress made in design and construction in the past several decades, focuses on the structural design of marine facilities for the berthing, mooring, and repair of vessels. This first chapter delineates the book's scope and purpose, providing a brief overview of marine civil engineering disciplines, the development and design of port and harbor marine facilities, and an overview of important literature and design guidance on the subject.

1.1 Scope and Purpose

This book primarily is concerned with the structural design of marine facilities for the berthing, mooring, and repair of vessels. It is intended to provide the civil and/or structural engineer with background information and general design requirements for port and harbor docking structures. It is not meant to be a step-by-step design guide or to cover basic structural design principles, but instead provides structural design criteria and addresses design problems peculiar to marine structures as they differ from traditional land-based construction. In this regard, the determination of design loads and environmental effects and the selection and proper application of suitable materials are emphasized. Because of the probabilistic nature of loads in marine structures, their magnitude, directions, and frequency of occurrence over the lifetime of the structure cannot be so accurately predicted, compared to land-based structures. The harsh marine environment contributes to more rapid deterioration, so the general design approach also differs. The book is organized to follow the general design process, with each chapter providing a general subject background, discussion of contemporary practices, and references for follow-up.

As the purpose of a marine facility is to service vessels, Chapter 2 presents a description of vessel design considerations and their types and characteristics. General design criteria are reviewed in Chapter 3, including site selection and layout, operational requirements, environmental conditions, and materials selection. Chapters 4 through 6 are concerned with the determination of design loads. Structure loadings caused by cargo storage and handling, vehicles and equipment, and superstructures and environmental loads acting directly on the structure, and other load sources are reviewed. Chapters 5 and 6 deal with berthing and mooring loads imposed by vessels, which often govern structural design. The design of fixed structures, such as piers, wharves, dolphins, platforms, and ancillary structures, is presented in Chapter 7. Chapter 8 is devoted to geotechnical considerations, emphasizing the importance of marine foundations and soil–structure interaction. Principles of the design of floating structures and their applications in marine facilities are covered in Chapter 9. An introduction to the design of dry docks, contributed by engineering specialists in the field, follows in Chapter 10. All port and harbor engineers should be familiar with the general dry dock types and their principles of operation. Chapter 11 deals with aspects of rehabilitation, maintenance, and repair and modes of deterioration. An understanding of structure maintenance problems and the need to consider future expansion or conversion provides the design engineer with important insights that can be incorporated into new designs as well. Chapter 12 is devoted to inspection methods and documentation, including an introduction to asset management. The book attempts throughout to emphasize basic principles applicable to vessels of all sizes, from yacht harbors to mammoth tanker berths, and from simple nearshore waterfront structures to offshore deepwater terminals.

The design of marine terminal and dry-docking facilities is only one aspect of the broader field of port and harbor engineering. Important related subjects not covered herein include port planning and economics; cargo handling and transportation systems; harbor siltation and dredging; and layout of navigation channels and design of breakwaters and shore protection structures. These subjects are already well covered in the general literature on port and harbor and coastal engineering, such as the references listed at the end of this chapter, which have been arranged according to specific subject areas. Not all of the general references listed have been cited in the Chapter 1 text. This book is intended to complement, update, and further contribute to the general body of textbook literature. An effort has been made not to duplicate material that already has been well covered, but rather to fill in some gaps and to provide guidance on the structural design process, as well as contemporary design standards and other important references. Given the breadth of the subject matter, any given chapter could be expanded into a book of its own, so the intent herein is to provide the necessary background over a range of subjects so that the reader can delve deeper into a specific topic or understand when specialized assistance should be sought.

A brief note about the usage of mathematical units is warranted. No attempt has been made to use entirely consistent units, such as the SI system; instead, the information is conveyed in the units in which it is usually encountered in practice in the United States, and for data reproduced from other sources, the units of the original presentation have sometimes been retained. In addition to metric and English units, marine work uses some units of its own, such as nautical miles, knots, fathoms, and others, and the port engineer should become accustomed to thinking in terms of the various units. Some important conversion factors and nautical units are included in Appendix 1.

1.2 Marine Civil Engineering

Marine civil engineering encompasses the planning, design, and construction of fixed and stationary moored floating structures along ocean coastlines, estuaries, lakes and rivers, and into the offshore realm. Traditionally, civil engineering in the oceans could be categorized as coastal engineering, dealing principally with shoreline processes and protection, and port and harbor engineering, dealing with the berthing, servicing, and navigation requirements of vessels. Today, another broad category—offshore engineering, concerned with the recovery of natural resources, namely, oil and gas, as well as alternative power projects that convert renewable energy sources such as wind, waves, and tidal currents—is an active branch of marine civil engineering.

Table 1-1 summarizes some of the types of civil engineering works involved in marine civil engineering broadly classified consistent with the Academy of Coastal, Ocean, Port and Navigation Engineers (ACOPNE) of ASCE's COPRI (Coasts, Oceans, Ports, and Rivers Institute) major subdisciplines as coastal, ocean, port, and navigation engineering. Contemporary uses for coastal and offshore areas, such as waterfront development for nontraditional maritime uses and ocean energy conversion, respectively, have been added to Table 1-1 as subdisciplines. The design of marine facilities is primarily within the domain of port engineering. There is necessarily some overlap of the disciplines as the design of breakwaters and harbor protection can be considered an aspect of port and harbor engineering as well. Another closely related field, navigation engineering, includes channel layout and dredging and the design of inland waterways, including canals, locks, and flood protection structures. Inland waterways provide a critical transportation link with major seaports. Although each of the disciplines noted serves a different purpose, all practitioners must cope with the same harsh environment and draw experience from each other. For example, developments in the study of wave forces and deepwater construction techniques conducted by the offshore oil industry have contributed to the technology base of traditional harbor works and have aided the development of marine terminals at deepwater exposed locations.

Historically, coastal and harbor works in the form of breakwaters and quays date back to antiquity. The earliest documented port and harbor works are attributed to

Table 1-1. Marine Civil Engineering Disciplines

	Coastal	Ocean	Port	Navigation
Purpose	Shoreline and harbor protection	Recovery of natural resources	**Vetting of vessels Cargo transfer**	Navigable waterways
	Erosion control			
Project type	Breakwaters	Fixed platforms	**Marine terminals**	Inland waterways
	Jetties and groins	Mobile drill rigs	**Piers and wharves**	Locks and canals
	Seawalls	Single-point moorings and moorings	**Harbor structures**	Channels and basins
	Revetments	Pipelines	**Shipyards**	Dredging
	Shore stabilization	Offshore terminals	**Dry docks**	Navaids/VTS
	Flood control		**Small craft harbors**	
	Outfalls, pipelines, and pollution control		**Moorings**	
	Artificial islands		Dredging	
	Coastal zone construction			
Related and subdisciplines	Nearshore monitoring	Ocean energy	**Offshore terminals**	Hydrographic surveying
	Hydrographic surveying	Buoys and monitoring	Waterfront development	
	Waterfront development	Exploration		
	Water quality			

Notes: Bold type indicates subjects addressed in this book. Navaids are aids to navigation, such as buoys and markers. VTS is vessel tracking system.

the early Egyptian and Phoenician cultures. The Romans possessed a certain genius for harbor construction, and remnants of some of their harbor works still exist today. They developed techniques for pile driving, constructed cofferdams, and most notably a durable pozzolan cement with which they built seawalls and harbor works by around 300 BC (Brandon et al. 2014). Inman (1974) and Jackson (1983) described ancient harbor engineering feats and their lessons for posterity, and Johnson (1974) reviewed the significant academic progress of more recent times. This book is concerned with certain aspects of harbor engineering that began to be studied as an applied science around the mid- to late-nineteenth century. It was during this period of industrialization that many of the early wave theories were developed, and early texts on harbor engineering were published (Stevenson 1864, Vernon-Harcourt 1885). As the era of canal building faded during the late nineteenth century, the first international conferences on vessel navigation in coastal ports began to focus on port development technology (PIANC 1985). From this time on, into the early twentieth century, harbor structures were built largely of timber or stone or composites of both. Vessel sizes began to grow, and shore transportation networks were well developed.

The introduction of reinforced concrete, iron, and steel into waterfront construction began what could be considered the golden age of harbor engineering works in the early- to mid-twentieth century. Greene (1917), Small (1941), and Clearwater (1985) provide insight into the early development of harbor engineering during the past century. In 1956, the American trucking entrepreneur Malcom McLean adapted a surplus tanker to carry containerized cargo on deck that could also be loaded onto truck trailers, and thus began the age of the containership that has revolutionized the intermodal transport of general cargo (Paine 2013). The general trend to larger and deeper-draft vessels and more sophisticated and faster material handling and transportation systems has continued. Today, more than 90% of world trade is carried by shipping; port facilities and infrastructure are the critical link to the distribution of goods produced worldwide. In the United States, the "Marine Highway," as defined by the U.S. Maritime Administration (MARAD), consists of more than 29,000 nautical miles of inland waterways, canals, bays and channels, and coastal and open ocean routes that link the marine routes with the country's transportation network. Information and statistical data on international shipping and maritime transport is published by the United Nations Conference on Trade and Development (UNCTAD 2015) and the International Maritime Organization (IMO 2012).

Port development and expansion kept pace with ship and material handling technology for a while, perhaps into the late 1960s or early 1970s, but in the United States and other developed countries, many port facilities now are in need of upgrading, rehabilitation, or replacement because of deterioration with age and obsolescence. The exponential growth of containership size within the past decade, for example, has many ports struggling to keep pace with the demand for larger and deeper berths with larger cranes and greater backland storage capacity. Today, we see the development of offshore ports and "megaports" to meet greater societal and shipping demands. Port development is strongly affected by political factors, such as world trade routes, regulation, lack or inaccessibility of suitable virgin sites, and environmental protection concerns. The waterfront has become an increasingly valuable resource, and conflicting uses such as urban and residential development have placed further pressures and limitations on suitable marine terminal sites. These issues are beyond the scope of this text, but the port and harbor engineer needs to be familiar with their implications and interfaces with marine facilities design.

Marine civil engineering draws heavily from other disciplines and often relies more on the application of engineering principles than on following strict building codes, compared with the design of most land-based facilities. The art and science of harbor engineering has been greatly affected by technological advances and activity in the following fields:

- Ship design and construction,
- Cargo handling and transportation systems,
- Dredging and construction equipment technology,
- Materials science,
- Applied oceanography,
- Mathematical and physical modeling techniques, and
- Surveying and electronic measurement.

1.3 Port and Harbor Facilities

Contemporary marine facilities tend to be highly specialized in their operations, which focus upon the type of cargo being handled. Marine terminals are active berthing facilities that serve as transfer points for the vessel's cargo. Major facility types include

- General (break bulk) cargo,
- Containers,
- Roll-on/roll-off (Ro/Ro),
- Ferries,
- Passenger and cruise ships,
- Dry bulk, and
- Liquid bulk and liquefied gases (especially liquefied natural gas [LNG] and liquefied petroleum gas [LPG]).

Multipurpose facilities may combine any of the above major types, such as general cargo, container, and Ro/Ro or both dry and liquid bulk. Other facilities may be even more specialized, such as for handling livestock or refrigerated cargo only, for example. Barge carrier ships can operate without specific shoreside berths but may moor alongside at general cargo or multipurpose facilities as they transfer their barges.

Permanent berthing facilities ("home ports") and shipyards are generally considered less active, in that vessels typically remain in berth for long periods. Such facilities include

- Military and Coast Guard bases,
- Shipyards (construction and repair),
- Fishing and commercial small craft,
- Research vessels, workboats, and offshore supply vessels (OSVs), and
- Small craft harbors and marinas (yachts and recreational craft).

Permanent berth facilities and most marine terminals are built near shore and are shore connected in relatively well-protected harbors. Liquid and dry bulk facilities, however, may be constructed offshore in relatively unprotected deep water. Their cargoes may be transferred to shore via pipelines or conveyor systems or may be transferred to small vessels, in a process termed *lightering*, for distribution to smaller coastal ports. Offshore ports may consist of vessels or floating structures known as FPSOs (floating production, storage, and offloading units) or FSOs (floating storage and offloading units) that are moored to single-point moorings (SPMs), which may also be equipped with hose connections for transferring liquid cargoes through underwater pipelines.

Fig. 1-1 shows an offshore "sea-island" berth for mammoth tankers of too deep draft and too large size to enter most conventional harbors. Such facilities must be designed to survive severe environmental conditions associated with long return periods and the berthing and mooring forces of mammoth vessels under maximum operating conditions. Petroleum products and liquefied gases are considered hazardous cargoes and are subject to strict regulations and fire protection requirements. Such facilities are often isolated from other waterfront activities, such as at the new Canaport LNG terminal shown in Fig. 1-2, which is constructed in deep water of an approximately 90 ft depth with up to 30-ft tide range and is exposed to the open waters of the Bay of Fundy, as described by Hebbale (2014).

All facility types have particular design requirements. Containership terminals generally require considerable backland area for the marshaling and storage of containers with intermodal connections and wharf structures designed to accommodate large cranes and heavy deck loads. The port of Vancouver, BC's Deltaport, shown in Fig. 1-3, is Canada's largest container terminal, designed as a high-volume, high-density facility with intermodal containers flowing directly to railcars from shipside. It has 3,609 ft of berth face with 10 container cranes and a low water depth alongside of 52 ft. Today's container ports are challenged by the rapid growth in ship size and capacity, thus requiring greater crane capacity and reach as well as container marshaling area (OECD/ITF 2015). The trend today is toward expansion of the largest container facilities to serve as load centers for the larger oceangoing vessels, with smaller vessels providing local coastal feeder service to smaller container port facilities, such as that shown in Fig. 1-4. The increase in ship size has a cascading

Fig. 1-1. An offshore "sea-island" type tanker berth. Crude oil transshipment terminal off Grand Bahama Island, Bahamas

Source: Photo courtesy of PRC Engineering, Inc./Frederic R. Harris, Inc.

Fig. 1-2. The Canaport LNG terminal in the Bay of Fundy, Saint John, New Brunswick, Canada, is capable of receiving the world's largest LNG vessels

Source: Photo courtesy of Canaport LNG

effect that also puts pressure on the smaller container ports to accommodate larger vessels that have been redirected from the major ports.

An important consideration in the planning and design of new marine terminals, especially at locations with limited waterfront space, is to consider multipurpose use. The model terminal (PIANC 1987) of today should be suitable for handling a mix of containers, general cargo, and Ro/Ro, as well as designed to handle the largest loads of

Fig. 1-3. The GCT Deltaport container terminal in Delta, British Columbia, Canada

Source: Photo courtesy of GCT, Global Container Terminals, Inc.

Fig. 1-4. A contemporary small- to medium-sized container terminal with dual container cranes. Note stacking area and control tower in background

Source: Photo by Paul Ubl, courtesy of Massachusetts Port Authority

today. Considering the long lifetimes of such infrastructure works and planners' limited ability to see very far into the future, this seems a prudent approach, especially for developing nations. PIANC (2014a) provides guidance for the development of master plans for the development of existing ports.

Fig. 1-5. A bulk cargo terminal facility equipped with a catenary continuous unloader, multipurpose Portainer crane

Source: Photo courtesy of Paceco, Inc.

Bulk terminals, such as that shown in Fig. 1-5, often feature elaborate material-handling equipment and conveyor systems. Space for stockpiling bulk materials must support very heavy loads. Ferry terminals, as shown in Fig. 1-6, require direct highway linkage and marine terminal structures with substantial fender systems and efficient transfer bridge and passenger boarding systems because of the rapid-turnaround nature of ferry service. Cruise and passenger ship facilities, similar to containership facilities, have been confronted with rapid industry growth in terms of ship size and number of passengers (Pallis 2015). Fig. 1-7 shows a contemporary cruise ship terminal with multiple berths and a spacious terminal building for accommodating a large volume of passengers on international voyages. Piers and wharves that support habitable structures and allow public access must be designed to meet the generally more stringent building code requirements for buildings as well as berthing, mooring, and cargo-handling loads.

Shipyard facilities require ample alongside berthing space with substantial utility and ship service systems and full revolving crane service. A central feature of ship repair yards, in particular, is dry docking capability (Chapter 10). Fig. 1-8 shows a contemporary shipbuilding facility consisting of three land-level building ways for modular assembly of ship sections and with selective transfer onto a floating dry dock launch platform that is translated to deep water for launching (see also Fig. 10-41).

Remote sites and underdeveloped areas, such as small islands and isolated military or research stations, require facilities that can handle a variety of cargo types, often with self-unloading vessels and piers and wharves that may be largely

Fig. 1-6. A passenger and vehicle ferry terminal with two ferry slips at Nantucket Island, Massachusetts. Note guide-in dolphins and wharf corner fenders for "warping" of vessel. Also note queuing areas for cars and tractor trailers

Source: Photo courtesy of the Maguire Group, Inc.

constructed off site in prefabricated pieces or floated into position, such as the jack-up barge pier in Thule, Greenland, located in the high Arctic, shown in Fig. 1-9 and described further in Section 7.1.

Large-scale offshore ports consisting of artificial islands constructed in deep water, such as the Khalifa Port and Khalifa Industrial Zone in Abu Dhabi, serve as pure "transshipment" ports and may represent a future trend (Pluijm 2014) where the offshore port serves as a terminus for very large vessels and cargoes are transferred to smaller vessels for local distribution to ports that cannot accommodate supersize vessels. The development of "megaports" often through the expansion of existing port facilities is driven by the advent of new megaships, megacities, and the facilitation of global economic demands. Megaports may be defined by large cargo volumes and throughput, economic value, and/or land and sea surface area used, (i.e., sheer size) (PTI 2016).

Port facilities may consist entirely of floating structures as described in Section 9.1. Fig. 1-10 shows a floating liquefied propane gas (LPG) FSO storage unit permanently moored to an SPM that serves as a floating port for the loading of LPG ships. SPM buoys have been developed to dock with purpose-built LNG and LPG vessels for onboard regasification and direct transfer to underwater pipelines at terminus locations.

Fig. 1-7. Canada Place Cruise ship terminal

Source: Photo by Colin Jewell Photography, courtesy of Port Metro Vancouver

Fig. 1-8. Bath Iron Works LLTF shipbuilding facility providing land-level transfer access to a floating dry dock from three separate building ways

Source: Photo courtesy of Greg Bridgman

Fig. 1-9. The world's most northern seaport at Thule, Greenland, constructed of float-in jack-up steel-hulled barges

Source: Photo courtesy of Appledore Marine Engineering, LLC

Specific facility-type requirements are discussed in greater detail in Section 3.3 and vessel types in Section 2.4. The design of all facility types in general must consider vessel berthing requirements, which often focus on one or more "design vessels," navigation channels and water depths, backland space requirements and shore transportation connections, site exposure and subsurface soil conditions, and other factors peculiar to the facility type. Specific information on existing port facilities are published in port directories, which often include data on piers, wharves, services, water depths, and climatic conditions by such organizations as the American Association of Port Authorities (AAPA), Drewry Shipping Consultants Ltd., IHS Lloyds, and MARAD; see Appendix 3 for website addresses.

1.4 Design of Marine Facilities

Marine facilities typically consist of piers, wharves, quays, bulkheads, fills, dolphins and platform structures, trestles and access bridges or catwalks, and buildings and backland civil and site works. The design of marine terminal buildings and civil engineering site works is the same as for any other land-based construction with due consideration to the harsh environment and the possibility of flooding and is not covered herein. The distinction between piers, wharves, and quays and the description of other fixed structural types is given in Section 7.1. "Sustainability" and the "greening" of port facilities is an important consideration that primarily affects port

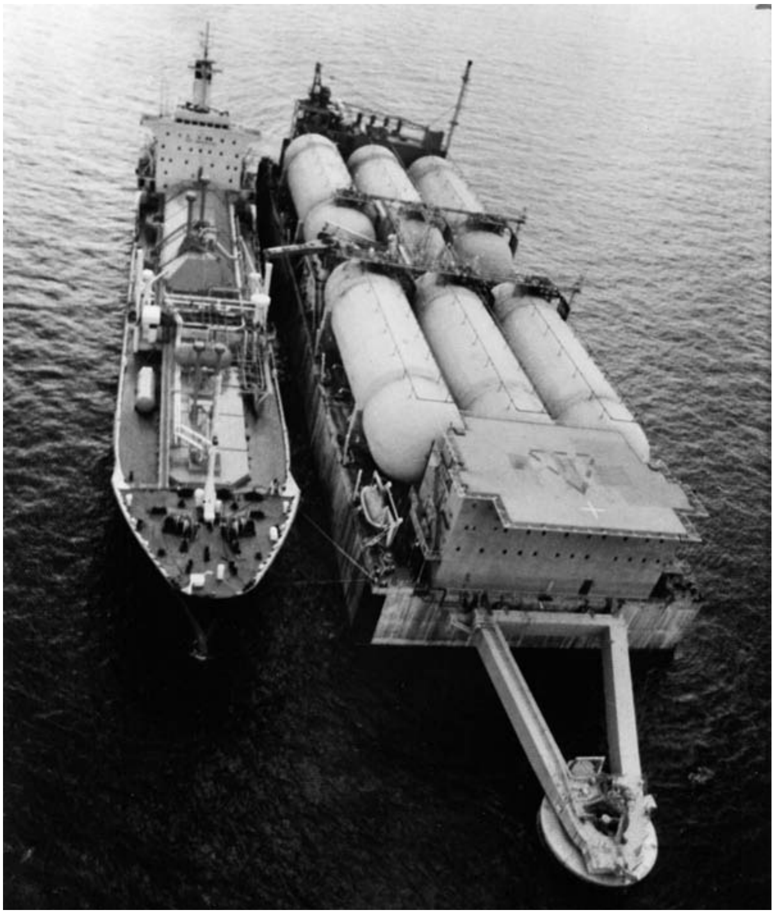

Fig. 1-10. A floating LPG terminal consisting of a 461 × 136 ft prestressed concrete hull with a gross displacement of 66,000 tons for LPG storage, moored to an SPM with yoke-type connection

Source: Photo courtesy of Concrete Technology Corporation

operations and terminal management practices (PIANC 2014b) but can be reflected in facilities design by designing for long-term durability and using environmentally favorable materials and construction methods.

The structural design process must consider all available materials and construction techniques with regard to both local site and environmental conditions and operational and vessel berthing requirements. The design procedure can be divided into key activities:

- *Establishment of design criteria.* Considerations include environmental and operational loadings and their probability and frequency of occurrence, as well as structural reliability as measured by factors of safety and allowable stresses and movements.

- *Determination of optimum structure type and configuration.* This activity includes preliminary design evaluation of alternative structures and construction methods and details.
- *Calculation of forces for design conditions.* These forces include combinations of short-term normal loads and long-term extreme loads.
- *Determination of structural response.* This step usually involves analysis of preliminary structural elements and arrangements on a trial basis until an optimum structure is found. Dynamic loads and structural response often form a feedback loop.
- *Comparison of structural responses with structural reliability criteria.* This step is done to meet minimum safety requirements and to ensure cost-effectiveness.

Unlike traditional land-based structures, which are designed for loads and allowable stress criteria established by building codes, marine structures are most often designed to loading and performance criteria established by the designer in concert with facility owner and/or operator requirements. A probabilistic approach is followed, considering the probability of occurrence, usually in terms of statistical return periods of specific events, and the structure's level of reliability under the specified conditions. For example, it often is assumed that large vessels will not remain alongside their berth when winds or wave conditions exceed a given threshold velocity or height, respectively. Cranes usually cease operation and/or loading arms are disconnected at even lower threshold values. Berthing and mooring loads often govern the lateral load capacity of structures considered herein, and vessel dimensions and berthing maneuvers affect the facility layout and structure configuration. Therefore, a maximum-design vessel under various conditions of ballast and water levels usually governs structural loads, whereas consideration of smaller or alternative vessel types dictates overall layout, number and locations of fenders, and so forth. In light of the foregoing discussion, it is therefore important to consider the likely simultaneity of occurrence of the various loads and load combinations.

Design criteria also must consider long-term effects, such as scour and erosion, general deterioration, fatigue and wear, construction techniques and weather windows, and the possibility of future expansion. The design life of a structure affects the material and durability design criteria. However, for most marine facilities, the physical life often exceeds the functional life because of obsolescence, the vagaries of the shipping industry, and changing socioeconomic conditions.

Although there are no definitive, binding building codes or standards that control the design of marine facilities, there are several guideline codes of practice to which the designer can refer for general design and for certain specific requirements as presented in the following section. In addition, specific criteria that have been established by international organizations relate to specific facility type requirements. These standards are introduced in Chapter 3, along with a more detailed discussion of general design criteria.

The evaluation of loads and forces acting on most nearshore waterfront structures can often be carried out with simple closed-end-solution equations and/or simple models using structural software programs. In deeper water and at sites exposed to significant wave action, more sophisticated analytical techniques may be required. Mathematical and/or numerical modeling techniques and/or physical hydraulic model studies may be used. In some instances, the results of such studies may be extrapolated and applied with caution to the design of similar structures under similar vessel and load conditions. Most waterfront structures can be analyzed using static methods, but offshore structures often require dynamic analysis. Under dynamic analysis, the forces and structural response become a function of each other and often lead to interesting results when compared with static analyses. Dynamic analysis, however, requires relatively detailed input information about subsurface soil conditions, the expected sea conditions, and vessel motion characteristics, as well as the elastic properties of the structure.

Reliability criteria are most simply expressed in terms of a factor of safety (FS), where

$$\mathrm{FS} = \frac{\text{Structure resistance (capacity)}}{\text{Applied load (demand)}}$$

The factor of safety may vary greatly according to material type, structure type, degree of confidence in and probability of exceedance of specified load conditions, and consequences of failure in terms of potential loss of life and/or economic loss. The FS typically is lower for long-term extreme events than for normal operation conditions. It is most important for the designer to consider all possible failure modes and examine the safety margin. For example, factors of safety cannot be applied directly to fatigue-loading conditions.

Marine structures should be designed for resiliency and should possess ample ductility and some form of redundancy or alternate load path to preclude catastrophic failure under extreme conditions and local yielding of structural elements. Probabilistic approaches, such as load and resistance factor design (LRFD) methods, are a rational means of dealing with the uncertainties of marine structures design. In LRFD design, the material resistance is reduced and the applied load is factored to provide the FS. LRFD criteria have been developed for offshore structures (API 1993, ACI 1984), which may be applicable to certain marine terminal and port structures located in deep water and at exposed locations where wave action is predominant. LRFD criteria for highway bridges (AASHTO 2014) may also be applicable to certain pier and wharf designs, especially where vehicular loads are significant in the design. Load factors for piers and wharves have been developed by the U.S. Navy (DOD 2005). In the allowable stress design (ASD) approach, the FS is inherent in code-specified reduced material stresses for a given type of applied load condition. In many instances, the marine design community has informally adopted its own, generally reduced, allowable stresses based upon experience and recognition of the uncertainties in loads and material performance. Comparing calculated stresses with an experienced designer's allowable stresses is a useful means of checking designs based upon LRFD or of existing structures that were designed under older, unknown codes or standards.

Waterfront structures traditionally have been designed with a high degree of conservatism, largely because of unknowns in possible loadings but also because of relatively rapid deterioration rates of structures in the sea. Today, with economic pressures, a better understanding of environmental loadings, and a statistical approach to determining design criteria (with high-speed computers to aid their analysis), as well as better quality control of materials and protective coatings and systems, designers have the ability to produce more economical designs on a rational basis. Engineers must be cautious when establishing design criteria, however, as the knowledge of marine loads and structure interaction is still incomplete, and the ability to accurately forecast future events remains limited.

1.5 Design Guidance and Information Sources

Great effort has been exerted in attempting to ensure that all references herein are up to date. However, with rapid advances in many fields and the rate of information exchange, it is likely that many references will be superseded over the useful life of this book. In fact, many outdated and out of print sources have been referenced throughout this book as they may contain useful information not found in the updated versions or they may provide helpful insights into the design of existing structures useful in upgrading or rehabilitation work. Also, most of the relevant older references should be familiar to many practicing engineers and available in company or technical and research libraries. The U.S. Navy NAVFAC design manuals (DM) and military handbooks (MIL-HDBK) series are a case in point. Many of these manuals, familiar to seasoned port and harbor engineers, have been converted to unified facilities criteria (UFC) published by the U.S. Department of Defense (DOD) (e.g., DOD 2005), and provide prescriptive design guidance common to all government services. Superseded versions may be referenced when they contain information not found in the current versions or are cross-referenced in other documents. The reader should confirm that he or she is using the latest information available, such as by accessing the appropriate Internet site as listed in Appendix 3. Many of the original U.S. Navy and Army design documents have been republished by Pile Buck, Inc., and may be available in print or CD-ROM format (see Appendix 3). Many professional and trade organizations publish journals and periodicals as well as sponsor specialty conferences that are important in keeping port and harbor engineers up to date on the latest technical developments and applications. Important journals, periodicals, and regular conferences are listed in Appendix 2, and Appendix 3 includes source websites for a variety of specific subject areas, including selected vendors, environmental data, materials, and general reference sources.

Important design guidance and codes and standards are listed in the following references. In the United States, the DOD UFCs are widely referenced by the marine design community even for nonmilitary projects. Of these references, UFC 4-152-01, *Design: Piers and Wharves,* is perhaps the most generally useful in marine facilities design (DOD 2005). Many other UFCs are referenced throughout this book in regard to specific topics. ASCE's Coasts, Oceans, Ports and Rivers Institute (COPRI) publishes several Manuals of Practice (MOPs) and certain standards that are also referenced throughout this book. As many port and harbor engineers get involved with international projects, and because other nations have developed important marine design standards of universal application, the marine facilities designer should become familiar with important international standards, such as the British Standards Institute (BSI), the German Society for Harbor and Foundation Engineering (EAU), the Overseas Coastal Area Development Institute of Japan (OCADI), and the Spanish Actions in the Design of Maritime and Harbour Works (ROM). The International Navigation Association (INA), generally referred to as PIANC for the "Permanent International Association of Navigation Congresses," publishes many important consensus reports by working groups (WGs) of its technical commissions, such as the Maritime Navigation Commission (MarCom), that provide valuable design guidance for a wide range of specific topics and are referenced frequently throughout this book. At the time of this writing, there are active PIANC working groups on design codes and standards for harbor structures, design of Ro/Ro (Roll on/Roll off) and RoPax (passenger vessel) terminals, and design of small to medium LNG terminals in addition to other groups and the many reports already published and referenced in this book. All career port and harbor engineers should become members of this important organization, as well as of ASCE's COPRI.

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Note that there are additional parts of the DOD UFCs and BSI and ROM codes that pertain to specific aspects of marine design and are referenced throughout this book.

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CHAPTER 2

Vessel Characteristics and Dimensions

The principal purpose of most marine facilities is the berthing and vetting of vessels, so it is essential that the ports and harbors engineer have a basic understanding of vessel design, construction, operating requirements, and, in particular, overall form and dimensions. Ship development and port development are interrelated, each at times influencing the characteristics of the other. This chapter first presents some important definitions and terms, followed by brief discussions of hull form and construction and a broad overview of vessel types and dimensions and salient features of many of the vessels currently in service.

2.1 Definitions

A vessel's principal dimensions are illustrated in Fig. 2-1. The vessel's *overall length* is designated LOA. The *length along its waterline*, which may vary somewhat with load condition, is designated LWL. The length between perpendiculars (LBP) is the length between the *forward perpendicular* (FP), where the ship's stem intersects the waterline, and the *aft perpendicular* (AP), usually the rudder post, where the vertical structure of the ship's hull intersects the *designed waterline* (DWL). The DWL is the nominal waterline at which a vessel is designed to operate, usually the full load condition. By international convention, ships are issued a load line assignment by classification societies, such as the American Bureau of Shipping (ABS), that limits the maximum draft to which they can be loaded with respect to route and season and freshwater versus seawater and other structural and stability considerations. The load line is displayed on the ship's side by an international load line or "Plimsoll " mark. The summer seawater load line is the primary load line normally assigned to the vessel and is demarcated by a bold horizontal line through a circle. Other horizontal lines above and below the circle indicate the maximum draft and hence minimum freeboard to which the vessel may be loaded for transit in various defined waters. For all practical port design purposes, the LBP and LWL are nearly the same for loaded vessels at their DWLs, and these labels are sometimes used interchangeably. For partially loaded or light vessels, the LWL is generally reduced. For most contemporary merchant ships, the LBP usually can be taken as approximately 95% to 96% of

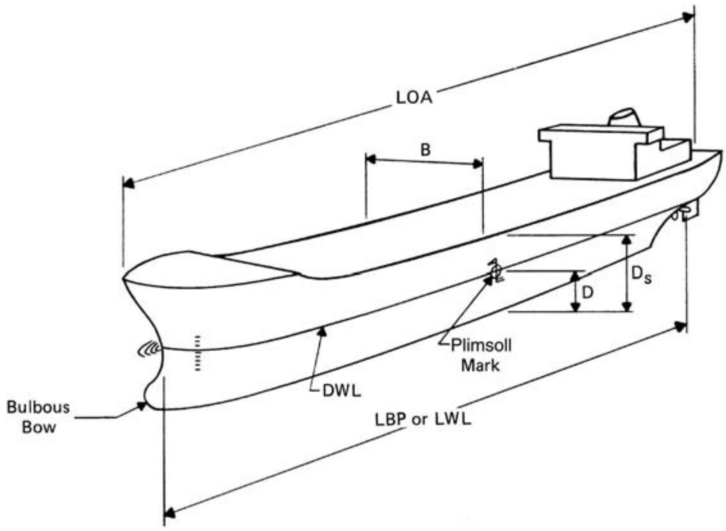


Fig. 2-1. Vessel dimensions definition sketch

LOA. Most contemporary oceangoing vessels have bulbous bows that project forward of the forward perpendicular below the waterline, in some cases for a considerable distance, so caution should be exercised in allowing for such projections. The *beam* (B) is the vessel's maximum width, which usually occurs at or near the vessel's transverse centerline or midship section. The depth of a vessel's hull (D_s) usually is measured at midship from the bottom of the keel to the top of the main deck (strength deck). Dimensions given on ships' drawings are usually "molded" dimensions referenced to the inside face of the shell plating. Molded dimensions of wooden vessels are usually given to the outside face of planking. The *draft* (D) is the distance from the vessel's waterline to the bottom of its keel or baseline. Naval architects typically use the symbol T to denote draft.

Many vessels draw more water aft than forward, so one must distinguish between draft forward, draft aft, and mean draft. For hydrostatic calculations, the mean draft must be used, and for navigation requirements, the maximum draft must be known. A vessel's draft increases approximately 2% to 3% in freshwater compared to seawater. *Trim* refers to the difference in drafts fore and aft, and *list* refers to a difference in drafts side to side. Certain vessels, such as many fishing and workboats, draw more water aft when on their DWL without being trimmed and are thus described as having a "raked" keel.

When a tanker or bulk cargo ship has discharged its cargo, it typically takes on ballast by flooding internal tanks with seawater in order to maintain stability and keep the propeller immersed, sometimes referred to as "sailing draft." This maneuver typically results in a significant amount of trim. Maximum ballast capacities of most vessels are on the order of one-half of their full load displacement, and a tanker "in ballast" is ordinarily at 30% to 50% of its full load displacement, depending upon weather conditions. Therefore, a vessel is usually somewhere between *lightship weight* (LWT), the weight of the ship without ballast and cargo, and full load condition for most berthing and mooring situations. For dry-docking, however, there usually is an attempt to get the vessel into LWT condition. *Freeboard* (FB) refers to the height of the vessel's main deck above the waterline; that is, $FB = D_s - D$. "Air draft" is the distance from the waterline to the top of the highest mast, which is important to know when there is an intervening bridge or overhead obstruction en route to a port facility. The total height from the bottom of the vessel's keel to the top of the highest mast (KTM) is especially useful to know for vessels that transit rivers or waterways with many bridges or overhead obstructions and at varying drafts. Another dimension peculiar to tankers is the bow to center of manifold distance (BCM), which needs to be known for the proper layout of tanker berths.

A vessel's total weight, termed its *displacement* (Δ), varies with its load condition from fully loaded, which is the displacement or *displacement tonnage* (DT) (the figure usually given, as in the tables in this book), to the weight at the lightship condition (LWT), which is the weight of the empty vessel, sometimes including minimal stores and partial fuel. Vessel displacement has traditionally been given in long tons (l.t.) of 2,240 lb; however, today, the metric ton (m.t.) of 2,205 lb is generally used. These ton measurements are nearly equal and equivalent to approximately 35 cubic ft (c.f.) of seawater (36 c.f. of freshwater). As a vessel displaces a volume of water equal to its own weight (Archimedes' principle), it is convenient to use 35 c.f./l.t. for converting from volume to weight. A vessel's displacement in volumetric measure, such as in cubic feet, sometimes is designated by the inverted delta, ∇. Note that short tons of 2,000 pounds are normally used for inland barges and some workboats.

The *stowage factor* refers to the relative density of the vessel's cargo, usually expressed in c.f./l.t. Tankers, for example, carry relatively high-density cargoes in the range of 39 to 46 c.f./l.t. and float low in the water, with the ship's *draft-to-hull depth ratio* (D/D_s) usually around 0.75. Tankers therefore undergo relatively large changes in draft and freeboard between loaded and ballast conditions. Containerships and roll-on/roll-off (Ro/Ro) vessels, by comparison, carry relatively low-density cargoes with storage factors in excess of 100 c.f./l.t. and D/D_s around 0.5, depending upon other specific design characteristics. These vessels undergo a relatively small change in draft and freeboard between loaded and unloaded conditions. Unit weights for various cargoes are presented in Section 4.1 of Chapter 4. Merchant vessels usually are referred to in terms of their cargo-carrying capacity, or dead weight (DWT), which is essentially the vessel's loaded DT minus its LWT. The actual cargo capacity equals the DWT minus the weight of the ship's crew, stores, and fuel. Dead weight tonnage is often expressed in kilotons (kDWT) for brevity.

For the purposes of figuring port duties, shipping costs, and so on, registered tons or tonnage is used. A registered ton is figured as 100 c.f. (2.83 c.m.) of internal space; a *gross registered ton* (GRT or more commonly GT) is equal to the ship's total internal volume, and a *net registered ton* (NRT or NT) is the total internal volume available for cargo (i.e., GT minus living, machinery, and fuel storage spaces). For most contemporary tankers and cargo ships, the LWT is on the order of 25% to 35% of the loaded displacement. The marine facility designer is primarily interested in the vessel's actual displacement and may need to know both the LWT and the DT or the displacement at a given draft. Displacement versus draft data are prepared by naval architects in the course of a vessel's design and are sometimes made available by the vessel's owner and/or operators.

Approximate relationships for determining vessel displacement tonnage, DT, from GT or DWT are given by OCADI (2009), PIANC (2002), and BSI (2000, 2013). There are some differences between the formulas given, and considering variations with particular vessel types and sizes, caution is advised in applying any of these formulas to a particular vessel. Conventional tankers and bulk carriers typically have a full load displacement from around 1.16 to 1.28 × DWT and very large crude carriers (VLCCs) from around 1.28 to 1.32 × DWT (ASNE/JMS 1999). It is generally sufficiently accurate to estimate a vessel's DT by applying a block coefficient to its principal dimensions, as described in Section 2.2.

A vessel's hull form, features, and general dimensions are of obvious importance to port engineers and are discussed further in later sections of this chapter. Knowledge of a vessel's hull structure is important in fender design applications and dry-docking. Other vessel characteristics of interest include vessel projected areas to wind and currents, which are addressed in Chapter 6.

2.2 Hull Form and Features

A ship's overall shape is defined on a *lines drawing*, or *lines plan*, such as illustrated in Fig. 2-2. The vessel's design and construction use molded dimensions, which may be measured to the inside or the outside of the vessel's shell plating, as specified in the plans. The vessel's shape is defined by various form coefficients, the most important of which is the block coefficient (C_B), which can be applied to the vessel's overall dimensions (in feet) to give the vessel's displacement:

$$\Delta = \frac{C_B \times \mathrm{LWL} \times B \times D}{35} = \frac{\overline{\nabla}}{35} \tag{2-1}$$

for Δ in l.t. in seawater. For freshwater, ∇ is divided by 36 c.f./l.t. instead of 35 c.f./l.t. Values of C_B normally range from 0.36 for fine, high-speed vessels to 0.92 for slow, full ships such as Great Lakes ore carriers (Lewis 1988). Naval combatant vessels have block coefficients typically in the range of 0.5 to 0.6. Table 2-1 gives the usual range of C_B for selected types of commercial vessels.

The length of parallel midbody (LPM), or the distance along a vessel's length over which the sides are essentially flat and vertical, is often of interest in fender system design and in berthing and mooring calculations. Ships with long parallel

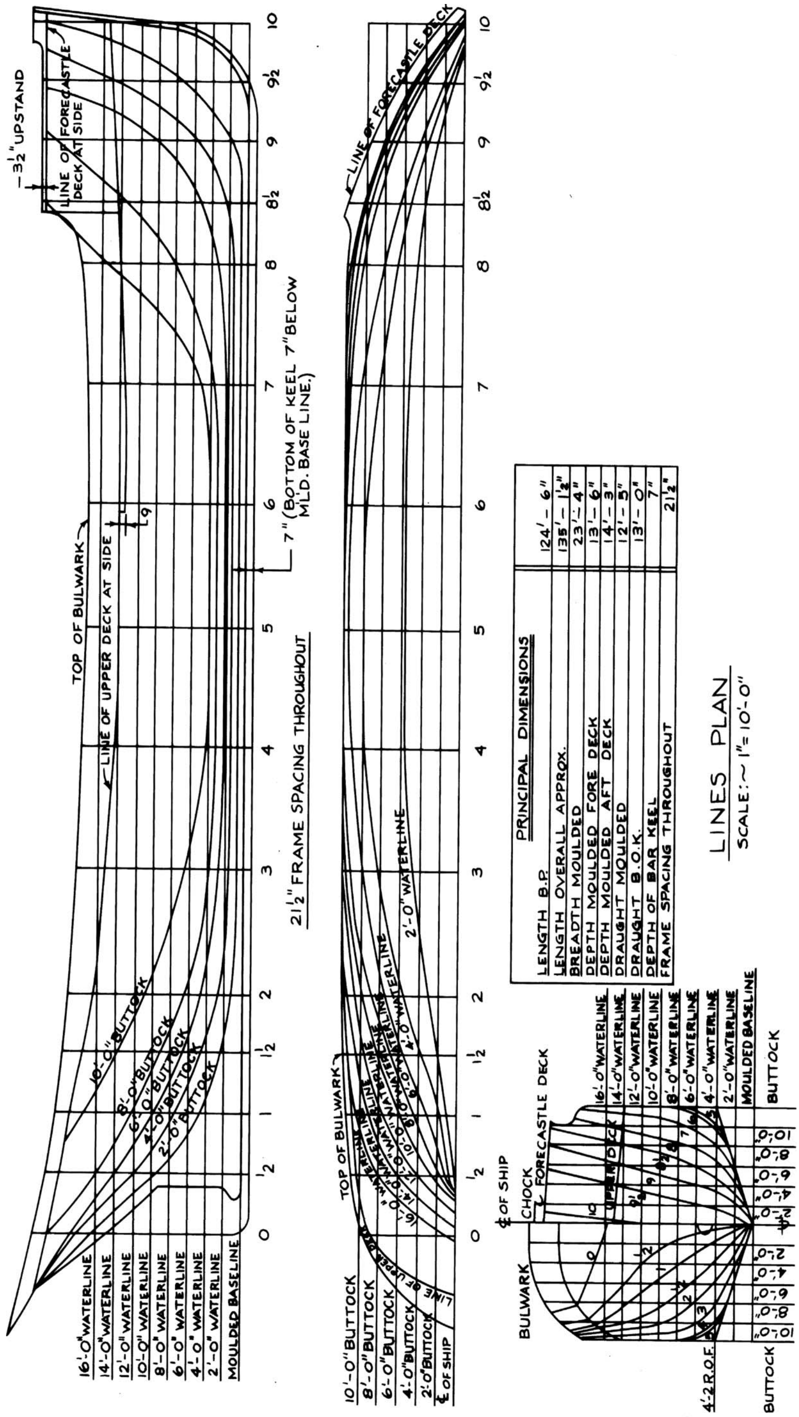


Fig. 2-2. Vessel lines drawing example

Table 2-1. Representative Block Coefficients (C_B)[a] for Selected Vessel Types

Vessel Type	C_B
Great Lakes ore carrier	0.88–0.92
Tankers:	
VLCCs/ULCCs	0.82–0.86
Product carriers	0.72–0.82
LNG	0.70–0.78
LPG	0.62–0.66
Bulk carrier	0.72–0.85
General cargo	0.60–0.75
Container ships	0.61–0.71
Barge carrier	0.58±
Ro/Ro (cargo)	0.70–0.80
Vehicle carriers	0.55–0.64
Ferries	0.55–0.65
Passenger vessels	0.45–0.65
Cruise ships	0.60–0.75

[a]Typical range for vessels in loaded condition.

midbodies, say 50% or more of the LOA, have relatively higher values of C_B. A general arrangement plan, when available, is most useful for locating mooring hardware and cargo holds, and equipment and can also be used to estimate the length of parallel midbody when a lines plan is not available. General arrangement plans for a variety of ship types can be found in Lamb (2003). Other important coefficients defining a vessel's form include the following:

- *Waterplane area coefficient* (C_{wp}), which varies with draft and is given by the actual plan area of the ship's hull at the waterline divided by LBP $\times$ B and is usually within the range of 0.7 to 0.9 for most large commercial vessels;
- *Midship section coefficient* (C_{ms}) is the actual transverse cross-sectional area of the hull at midship divided by $B \times D$, which is usually between 0.9 and nearly 1.0 for most large commercial vessels; and
- *Prismatic coefficient* (C_{pr}) is the displaced volume divided by the volume of a prism equal to the midship area × LWL, so that $C_{pr} = C_B/C_{ms}$, assuming that LWL = LBP. C_{pr} is a measure of the fineness of the ship's underwater hull form and may be important in ship motion studies.

Typical values of coefficients of form for various naval and commercial vessels can be found in ASNE/JMS (1999) and in the naval architecture texts referenced subsequently. The *vertical center of gravity* (v.c.g.) for most large commercial ships is usually within the range of around 0.6 to 0.75 times the hull depth (D_s) above the keel for ships in LWT condition. Most naval ships have a v.c.g. between 0.55 and 0.63 D_s

in the full load condition. For large ships floating on level trim, the *longitudinal center of gravity* (l.c.g.) is generally within about $0.02 \times \text{LWL}$ from midship and the l.c.g. can generally be assumed at midship for most port engineering applications. The amount a vessel sinks or emerges when weight is added or removed is given in terms of *tons per inch immersion* (TPI), and the *moment to trim 1 inch* (MTI) gives the change in trim when weights are shifted fore or aft. Means of calculating TPI and MTI and other hydrostatic and stability parameters such as *metacentric height* (GM) are presented in Section 9.2 of Chapter 9.

Dimensional Relationships

A vessel's general proportions can be described in terms of certain dimensional ratios of interest to harbor engineers, such as the vessel *length-to-beam ratio* (L/B), *beam-to-draft ratio* (B/D), and *length-to-draft ratio* (L/D). Almost all vessels lie within the following approximate limits (Lewis 1988): $L/B = 3.5$ to 10, $B/D = 1.8$ to 5, and $L/D = 10$ to 30.

In the above ratios, L is usually taken as the LBP. Most commercial merchant vessels have L/B ratios in the range of 5.5 to 7.0 and B/D ratios on the order of 2.7 to 4.0. Certain vessels such as ocean liners and Great Lakes ore carriers have a relatively high L/B on the order of 9.0 to 10.0, and European river cruise vessels may go over 12.0 or more, whereas for workboats, fishing boats, and certain automobile ferries, L/B is often in the range of 3.5 to 4.5. A vessel's proportions are a direct consequence of its function. For example, compared to containerships, break bulk cargo ships are slower and typically have lower L/B ratios; and because they are generally smaller and have fewer stability problems due to containers on deck, they have lower B/D ratios as well. Ro/Ro vessels have lower L/D ratios than containerships because they need the extra hull depth to accommodate the trailer chassis. Tankers vary considerably in their dimensional ratios, but the larger vessels tend to have a moderate L/B in the range of 5.5 to 6.5, and, being slower, they also have relatively high block coefficients. More rigorous treatment of vessel hull form and design features can be found in naval architecture textbooks, such as Letcher (2009), Lamb (2003) and Gillmer and Johnson (1982) and in the technical literature of the Society of Naval Architects and Marine Engineers (SNAME) and the Royal Institute of Naval Architects (RINA).

Dimensional Constraints and Size Classifications

Other factors affecting vessel proportions include physical constraints imposed by the vessel's trade route, such as water depths and canal or waterway widths. For example, the limiting lock width of the Panama Canal is 110 ft with a length of 1,000 ft and depth of 41.5 ft so that the maximum beam for a vessel that can currently transit the Panama Canal is 105.75 ft, resulting in the so-called "Panamax" size vessel. The Panama Canal is currently being widened to 180 ft with a length of 1,400 ft and a control depth of 60 ft and "Post-Panamax" or "new Panamax" (NPX) vessels are being constructed or are already in service (Stagg 2011). The NPX vessels have a beam of approximately 160 ft and up to about 1,200-ft LOA, corresponding to a containership of about 12,000 20-ft equivalent unit (TEU) capacity (Tseng 2011) or a contemporary cruise ship of about 90,000 GT. For an in-depth analysis of the Panama Canal widening and the development of Panamax vessels and effects on ports, see Payer and Brostella (2006). "Handymax" generally applies to dry cargo vessels less than around 50 kDWT. Vessels transiting the Suez Canal are limited to a 185-ft beam or less depending upon additional limitations on length and draft and other navigability requirements, resulting in the "Suezmax" moniker. The Saint Lawrence Seaway limits a vessel's beam to about 78 ft, which is sometimes referred to as "Handysize" or "Seawaymax," corresponding to from 28 to about 40 kDWT. "Capesize" applies primarily to bulk carriers required to transit between oceans via the Cape of Good Hope or Cape Horn because of their large size, generally around 150 kDWT or larger. "Chinamax" also applies to bulk carriers up to 400 kDWT as limited by Chinese port infrastructure, and "Malaccamax" refers to the maximum size vessel that can transit the Straits of Malacca because of navigational restrictions. "Aframax" vessels are a consequence of regulatory restrictions referring to the "Average Freight Rate Assessment," and the name applies only to tankers limited to from 75 to 115 kDWT maximum.

A ship's air draft may restrict it from entering certain harbors. For example, cruise ships with especially high superstructures may be prevented from entering important ports, such as New York, San Francisco, and Vancouver in North America, because of the Verrazano–Narrows, Golden Gate, and Lions Gate bridges, respectively, which have vertical clearances in the range of 197 to 223 ft. The megaship *Oasis of the Seas* is fitted with retractable exhaust pipes to reduce its air draft to around 213 ft when needed (Levander 2011a). The Port Authority of New York and New Jersey is planning to raise the deck of the Bayonne Bridge from its original 151-ft clearance above mean high water (MHW) to 215 ft to accommodate New Panamax class containership traffic (Petroski 2015). Environmental regulations also may affect a vessel's hull form and dimensions, such as restrictions imposed by double-bottom, double-hull, and segregated ballast requirements for tankers and certain barges and bulk carriers.

Vessel Features

The various vessel types may exhibit particular features that need to be considered in the design of marine facilities, such as the bulbous bow structures that protrude below the waterline and may strike the dock structure, especially at steep angles of approach if not accounted for in the dock design. Cruise ships and Ro/Ro vessels often have bridge wings that extend beyond the vessel's side and could strike dockside equipment. Many of the largest cruise ships have superstructures that are wider than the vessel's hull. Containerships and cruise ships typically have a definitive flare to their bow geometry that may be problematic to proper fender system contact. Many vessels, ferries and Ro/Ros in particular, have reinforced belting along their sides, generally at the main strength deck level, that affects fender system design. Certain vessel types, such as car carriers and cruise ships, have high freeboard and superstructure heights, resulting in large wind exposure areas, and loaded barges may have extremely low freeboards that are problematic in fender system layout and design. Various vessel types may have protruding loading doors and/or gangway structures. Military vessels may be fitted with sonar domes, propeller guards, and fins or strakes, all of which need to be accounted for by the marine facility designer.

High-Speed Craft Considerations

High-speed craft (HSC)—including catamarans, small-waterplane-area twin hull ships (known by the acronym SWATH), and various special-purpose military and law enforcement vessels—are usually constructed of aluminum and are more susceptible to impact damage than steel-hulled vessels. They also typically are of peculiar proportions, such as the much higher B/L ratios of catamarans, and may also have vulnerable appendages, such as hydrofoils, control fins, and water jets. HSC ferries generally are belted.

2.3 Hull Construction

The vast majority of vessels of interest herein are constructed of structural steel, although smaller vessels may be constructed of aluminum, wood, fiberglass-reinforced plastics (FRP), or even reinforced concrete or "ferro-cement." The following discussion is mostly confined to steel hull construction. The vessel's hull can be thought of as a box girder made up of stiffened plate elements that form the top deck (also called the weather deck or strength deck), which is the girder's top flange, plus bottom shell plating (bottom flange) and side shell plating. There are two basic distinct types of overall framing arrangements: transverse and longitudinal framing, as illustrated in Fig. 2-3. Both types use transverse frames and bulkheads with longitudinal beams and stringers. The difference between them is that in the *transverse system*, the frames are continuous and closely spaced, with longitudinal girders that are deeper and spaced farther apart than in the *longitudinal system*, which has closely spaced but shallower longitudinals with widely spaced deep web frames. Most vessels use a combination of these systems, typically with longitudinal framing of the deck and bottom and transverse framing of the sides for a more efficient hull structure. The construction of any given ship, military vessels especially, may lie anywhere between these two extremes. Side shell hull plating thickness varies with ship type and stringer spacing but is generally around 1/2 in.+ to 7/8 in.±, with spans between longitudinal stiffeners or "stringers" around 2.5 to 3.0 ft for ocean-going vessels. Transverse frame spacing is often within the range of approximately 10 to 20 ft. The term *scantlings* refers to the dimensions of a ship's structural members. The American Bureau of Shipping's *Rules for Building and Classing Steel Vessels* (ABS 2015a) gives minimum scantling requirements for ship hull design.

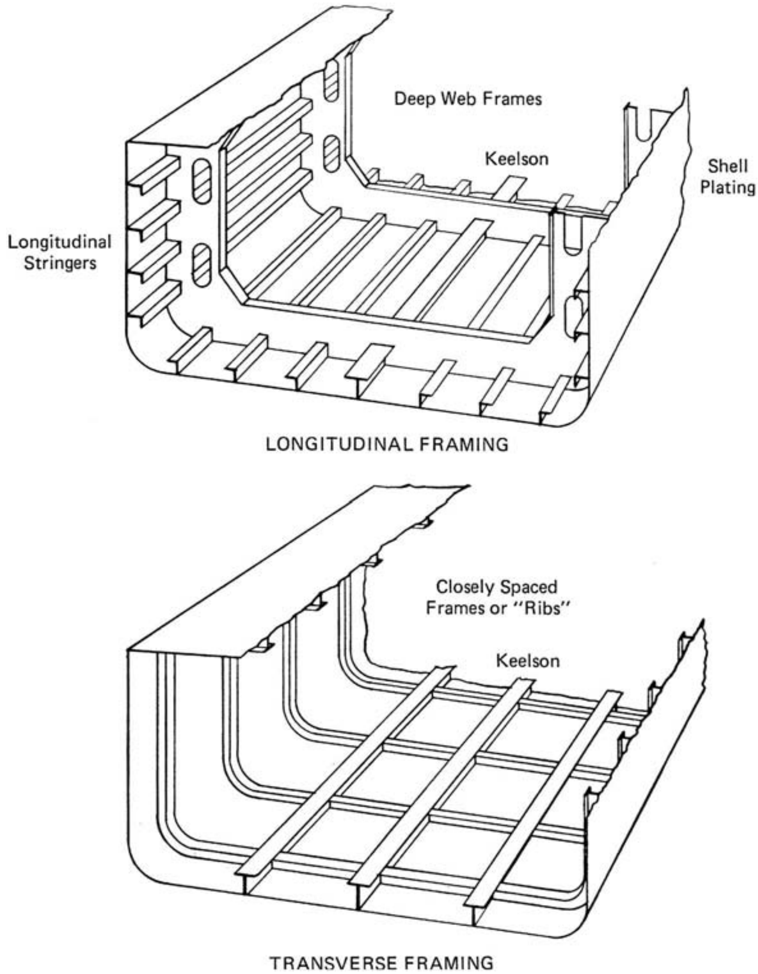


Fig. 2-3. Ship hull framing types schematic

The most important stresses that the overall hull girder must resist arise from the longitudinal bending moment and torsion induced by waves and by cargo and ballast distribution. When a ship is supported on a wave with a length nearly equal to the vessel's LWL, the vessel is in a *sagging* condition when the wave crests support its ends and in a *hogging* condition when supported amidships by a wave crest. The longitudinal strength and stiffness of floating structures are discussed further in Section 9.4 and in Hughes and Paik (2010), Mansour and Liu (2008), and Lamb (2003). Locally the ship's hull plating must resist hydrostatic and dynamic pressures plus the combined stresses induced through its overall girder action. The ability of the hull to distribute and resist localized loads (i.e., allowable hull pressure) is very important in designing fender systems and is discussed further in Section 5.6.

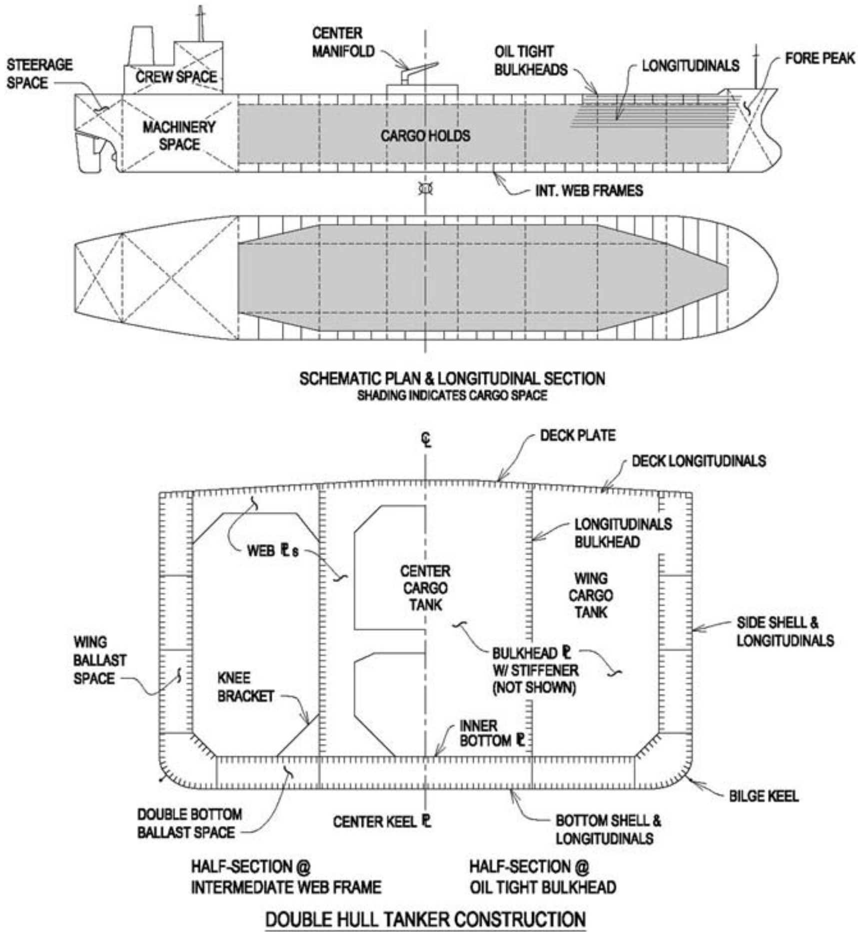


Fig. 2-4. Double-hull tanker schematic framing plan and sections

Tankers and certain barges and bulk carriers constructed subsequent to the 1990 Oil Pollution Act (OPA) are required to have double-hull construction in accordance with the International Maritime Organization (IMO) regulations. This construction results in an effective inner hull that contains the liquid cargo with void space that can be used for ballasting between the outer-hull shell plating. A schematic midship section with plan and longitudinal section of a representative double-hull tanker framing arrangement is shown in Fig. 2-4. Tankers of greater than around 75 kDWT generally have two longitudinal bulkheads, as shown in Fig. 2-4, whereas smaller tankers typically have only one along the centerline and the smallest tankers may have none. Bulk carriers are generally of similar construction to tankers but usually have double-hull bottoms and are often transversely framed along the sides.

2.4 Vessel Types and Dimensions

As of January 31, 2013, there were an estimated 28,170 oceangoing ships of 1,000 GT or greater of the top 25 flags of registry in the world. Table 2-2 gives a statistical breakdown of the number and tonnage of the top 25 world merchant fleets by vessel type and country of owner for vessels of 1,000 GT and over, taken from the U.S. Maritime Administration (MARAD) website (see Appendix 3). The world fleet of global merchant shipping, including all vessels of 100 GT and more, consisted of 89,464 active commissioned vessels as of early January 2015, amounting to 1.75 billion DWT, according to the United Nations Conference on Trade and Development (UNCTAD 2015), with an average vessel age of 19.9 years. World fleet statistics are also published by the International Maritime Organization (IMO) (see Appendix 3 for web addresses). Merchant ship statistics are published annually (Lloyds 2014), giving a detailed analysis of vessel size, type, tonnage, age, and so on, by country of registry, and Clarkson's Research also publishes a register of ships by ship type as well as a host of other shipping information (Appendix 3).

In the United States, the Merchant Marine Act of 1936 began the construction of the "C" series cargo ships and "T" series tankers, and a few of these vessels, built in the post–World War II boom, are still active. Dillon et al. (1976) present a detailed listing of vessels built under the Maritime Administration's (MARAD) Merchant Marine Act of 1936, which includes dimensional and other data of interest for vessels built under the act up until 1976. In the mid-1950s, containerized cargos were introduced, and they became prominent in international trade during the 1960s. In the late 1960s and early 1970s, roll-on/roll-off (Ro/Ro) vessels came into vogue, especially on the shorter voyage routes. Lighter-aboard-ship (LASH) barge-carrying-type vessels also came into being along ocean routes that service inland waterways. *Lightering* is the process of transferring cargo from a larger vessel to a smaller one that can enter shallower and/or more restricted waterways than the larger vessel. Tankers grew from the World War II vintage T-2 class of 16,700 DWT and 524-ft LOA to more than 500,000 DWT and 1,300-ft LOA. Fig. 2-5 depicts two such vessels in the process of lightering. The historical development of the U.S. Merchant Marine is described by Whitehurst (1983).

Vessels cover a wide range of sizes and shapes, from the smallest yachts and harbor craft to supertankers and aircraft carriers. Trade routes, port development, commodities, and cargo-handling methods may have a profound effect on ship configuration. There are too many specialized types of craft to describe in a text such as this; instead, a review of the most common types is presented, along with follow-up references. Although representative data for various ship types are useful for instruction and comparison, the designer always must gather specific information on the particular vessels to be served at a given facility. Vessels can be broadly categorized with regard to their mission, as commercial vessels, consisting primarily

Table 2-2. Top 25 Fleets of Privately Owned Vessels[a], by Vessel Type and Country of Owner, as of January 31, 2013

	Total			Containership			Dry Bulk			General Cargo			Roll-On/Roll-Off			Tanker		
Flag of Register	No.	GT (000)	Dwt (000)	No.	GT (000)	Dwt (000)	No.	GT (000)	Dwt (000)	No.	GT (000)	Dwt (000)	No.	GT (000)	Dwt (000)	No.	GT (000)	Dwt (000)
Panama	6,158	210,661	328,206	706	34,295	38,105	2,604	108,980	199,557	1,103	7,802	10,846	414	15,612	5,970	1,331	43,971	73,728
Liberia	2,974	124,207	193,408	1,010	40,834	47,839	785	34,726	63,205	235	2,883	3,779	34	1,298	454	910	44,467	78,132
Marshall Islands	1,876	82,484	136,328	237	7,351	8,700	753	32,478	59,229	92	1,404	1,894	22	869	309	772	40,381	66,196
Hong Kong	1,855	76,474	127,077	330	15,773	17,910	930	41,845	76,864	219	3,183	4,476	14	308	159	362	15,365	27,668
Singapore	1,776	59,814	92,513	378	13,323	15,451	349	16,871	31,191	121	1,881	2,473	76	3,224	1,242	852	24,514	42,156
Bahamas	1,029	42,430	63,125	57	1,553	1,748	256	8,389	14,794	247	3,094	4,245	75	3,526	1,266	394	25,868	41,072
Malta	1,587	42,165	69,065	130	5,306	6,015	549	19,130	34,527	321	1,732	2,356	50	1,274	664	537	14,723	25,503
Greece	704	39,909	72,480	33	2,196	2,392	250	12,897	24,289	28	49	65	10	127	79	383	24,639	45,655
China	1,416	30,780	49,384	160	4,986	5,985	553	16,094	27,898	286	1,955	2,662	28	341	132	389	7,404	12,707
Cyprus	788	19,580	31,438	208	4,090	4,894	290	10,398	18,781	153	993	1,344	14	213	84	123	3,885	6,335
Japan	577	15,780	22,696	2	101	101	183	7,047	12,988	37	185	243	108	2,190	884	247	6,258	8,480
United Kingdom	397	15,145	17,661	153	9,103	9,842	37	2,252	4,219	75	415	580	52	1,968	743	80	1,407	2,277
Isle of Man (British)	332	13,813	23,343	8	739	743	80	5,241	9,947	56	377	540	11	143	46	177	7,312	12,067
Italy	483	13,753	20,254	15	710	820	111	4,666	8,624	45	366	502	70	2,641	1,329	242	5,370	8,980
Norway (NIS)	383	13,236	17,657	0	0	0	66	2,504	4,358	59	1,160	1,634	48	2,587	1,065	210	6,985	10,600
Germany	288	12,716	14,622	218	11,965	13,518	2	158	326	41	218	269	3	50	21	24	325	487
Antigua & Barbuda	1,221	10,672	13,798	390	5,359	6,723	40	916	1,534	751	4,042	5,241	19	225	131	21	129	169
South Korea	764	10,634	17,232	82	808	1,053	180	6,767	12,475	259	1,147	1,576	22	433	231	221	1,478	1,897
Denmark (DIS)	307	10,596	13,373	90	6,570	7,430	4	204	401	34	65	82	12	328	131	167	3,429	5,328
Indonesia	1,396	8,650	12,856	156	1,108	1,461	120	1,712	2,894	671	1,968	2,770	51	234	133	398	3,628	5,597
Bermuda (British)	118	8,597	10,874	17	676	745	26	2,062	3,986	4	55	56	0	0	0	71	5,804	6,086
United States of America[b]	187	7,073	7,902	75	3,079	3,357	6	159	260	22	192	238	34	1,611	692	48	2,010	3,340
Malaysia	269	6,087	8,188	23	170	225	8	150	258	66	233	316	13	149	74	159	5,386	7,315
Turkey	546	6,059	9,388	44	592	747	105	2,875	4,978	261	921	1,419	24	508	241	112	1,162	2,003
Netherlands	749	5,453	7,171	57	927	1,084	9	403	693	575	3,176	4,353	19	367	229	89	580	812
Top 25 Registries	28,178	886,744	1,380,025	4,579	171,613	196,889	8,296	338,926	618,276	5,761	39,497	53,959	1,223	40,229	16,310	8,319	296,479	494,591
Total	36,307	975,176	1,508,939	4,909	179,875	206,547	9,307	360,301	654,966	9,908	55,226	74,617	1,684	47,480	20,163	10,499	332,294	552,645

[a]Oceangoing self-propelled, cargo-carrying vessels of 1,000 gross tons and above.
[b]Includes two integrated tug barges. There were vessel additions to and removals from the U.S.-flag fleet during 2013, the most current data posted. For the latest U.S.-flag fleet, please visit: http://www.marad.dot.gov/library_landing_page/data_and_statistics/Data_and_Statistics.htm. http:/www.marad.dot.gov/documents/USFlag-Fleet.xls.

Fig. 2-5. Two tankers engaged in lightering illustrate the dramatic increase in tanker sizes from WW II vintage T-2 to contemporary VLCC

Source: Photo by Lloyd Koenig, courtesy of Seaward International, Inc.

of merchant ships engaged in trade and the transportation of cargoes and/or passengers; industrial vessels, engaged in specific tasks or operations, such as drill ships, research vessels, dredgers, fishing vessels, and so on; service vessels, such as tugs, pilot boats, fireboats, and so forth; and military vessels. As a rule, commercial-type vessels are of the greatest general interest to port and harbor engineers, and most of the following discussion focuses on the most common merchant vessels. Commercial vessel data are continually updated in Lloyds Register (Lloyds 2014), and PIANC (2014, 2002) include tables of typical ship dimensions over a range of sizes within prescribed confidence limits for various ship types.

Tankers are liquid bulk product carriers (crude oil is by far the most common cargo), although various chemicals and other refined products may be carried by specialized or multipurpose tankers, often referred to as *product carriers.* Virtually all tankers over 100,000 DWT (supertankers) are crude oil carriers. Tankers have established the state of the art in ship size; vessels of 550,000 DWT and more than 1,300-ft LOA have been built. The largest ship built to date is actually an offshore floating storage unit (FSO) at 565 kDWT and just more than 1,500-ft LOA. The largest tanker built at 550 kDWT has been retired from service, and currently the largest tankers have settled in at a more practical size limit of around 350 kDWT. Local feeder lines may operate coastal tankers of 2,000 DWT or less, although smaller shipments are usually transported by barge. Tanker capacities, especially for smaller vessels, are sometimes given in barrels (bbl), where 1 bbl = 42.0 U.S. gallons. The crude oil capacity of a ship can be approximated by multiplying its

DWT by 6.5 barrels per short ton. Tankers can be classed by size for convenience, as follows:

1. Conventional tankers are generally less than 100,000 DWT and 40 ft or less draft. Such vessels usually are operated in domestic and/or coastal trade routes and can enter almost any major seaport. Tankers more than approximately 25,000 DWT to 50,000 DWT up to around 150,000 DWT are generally referred to as supertankers and have loaded drafts in excess of 40 ft, but they may enter certain U.S. ports partially loaded. Tankers of less than approximately 15,000 DWT are usually coastal tankers. General-purpose product carriers, including specialized parcel tank ships that carry liquid chemicals in special segregated tanks or containers, may be small or "handy" sized up to about 35,000 DWT, and midsized product carriers that may also carry crude oil range up to around 75,000 DWT.
2. VLCCs, or very large crude carriers, are generally between 150,000 DWT and 300,000 DWT with loaded drafts from 60 to 80 ft. This size is the workhorse of the general overseas trade; 200,000 DWT to 250,000 DWT are especially common. These vessels may enter some deep-draft ports, but they more commonly load and offload at offshore terminals and/or by lightering, transferring their cargo to smaller vessels.
3. ULCCs, or ultralarge crude carriers, weigh more than 300,000 DWT and have drafts of 80 to 100 ft. Although vessels of 1,000,000 DWT technically are feasible and had at one time been forecast by the year 2000, the practical economic limit has already been reached, or perhaps exceeded for the foreseeable future. Such very large vessels are strictly relegated to offshore terminals or storage systems.

There is no strict technical definition of VLCC, ULCC, or supertanker; the above descriptions represent a rather loose classification. All new tankers built since 1990 and many tankers over 70,000 DWT have double hulls as a result of the 1973 Intergovernmental Maritime Consultative Organization (IMCO now known as IMO) pollution control regulations referred to as MARPOL 73/78 (IMO 1973) and the requirement that such vessels have segregated ballast tanks. The general effect of this requirement has been to increase the ship's hull depth, D_s, and increase freeboard rather than draft. Large tankers typically are longitudinally framed to better cope with large sea bending moments caused by their long lengths. Table 2-3 gives specific dimensions for a popular size, representative double-hulled product tanker capable of entering most major U.S. harbors. VLCCs and ULCCs are generally relegated to deepwater offshore terminals. Fig. 2-6 shows the range of LOA, B, and D versus DWT for tankers. Detailed information on specific tankers and barges may be available through the Oil Companies International Marine Forum's (OCIMF) Ship Inspection Report Programme (SIRE) via the ship owner or terminal operator (see Appendix 3 for web address and additional vessel information sites). Contemporary tankers of 100,000 DWT and above usually have L/B ratios within the 5.5 to 6.5 range, compared with around 7.5

Table 2-3. Principal Dimensions and Design Data for Representative Mid-Size Double-Hulled Product Tanker

$LOA = 599$ ft	$GT = 21{,}809$
$LBP = 567.3$ ft	$NT = 14{,}774$
$B = 97.1$ ft	$C_B = 0.776$
$D_s = 51.2$ ft	$L/B = 5.84$
$T = 35.8$ ft[a]	$B/T = 2.79$
$T_{bal} = 14.4$ ft fwd/23.1 ft aft ("sailing draft")	
$T_{lwt} = 2.6$ ft fwd/15.4 ft aft (at LWT)	
$\Delta = 43{,}527$ mt[a,b]	
$DWT = 33{,}370$ mt[a]	
$LWT = 9{,}796$ mt	

[a]At assigned summer draft load line.
[b]m.t.=metric tonne

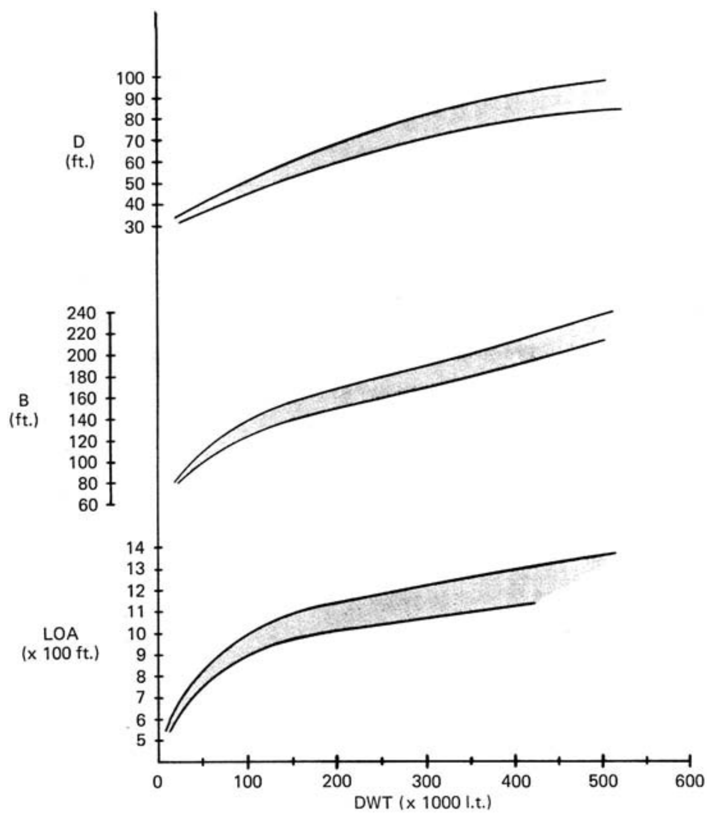


Fig. 2-6. Typical range of tanker dimensions versus DWT

for smaller, earlier tankers. Block coefficients are in the 0.80 to 0.84 range, compared with around 0.75 for earlier vessels.

Gas Carriers include **LNG** (liquefied natural gas) and **LPG** (liquefied petroleum gas), primarily; propane, butane, butadiene, propylene, and ammonia carriers are distinct specialized classes of vessels. LNG hulls cradle heavily insulated cryogenic tanks that contain liquefied gas at very low temperatures, typically at near-atmospheric pressure. The larger and most recent ships typically have membrane-type tanks that are integral with the hull structure, and earlier designs and mid to smaller size ships may have spherical or prismatic tanks isolated from the hull structure. LNG must be cooled to −285°F, at which point it liquefies and occupies a volume 1/600 of its gaseous volume. The capacity of such vessels usually is given in cubic meters (c.m.). Most contemporary ocean-crossing vessels are in the 125,000-c.m. plus range and have a more than 900-ft LOA. The current trend seems to be for larger vessels at offshore ports. The Qatar gas Q-Max class is the record holder at 1,132-ft LOA and 266,000-c.m. capacity. Until the early 1970s, vessels in the 40,000 to 75,000-c.m. range were most common. LPG carriers may be liquefied at high pressure, low temperature, or a combination thereof and thus may be fully pressurized or fully refrigerated or semipressurized, depending somewhat on size and specific cargo. Fully pressurized vessels require thick steel tanks and are typically smaller with typical capacities in the 3,000 to 7,500-c.m. range. Fully refrigerated vessels operate at atmospheric pressure and around −45°F for propane, typically from around 15,000-c.m. up to around 85,000-c.m.capacity. Partially pressurized vessels range up to around 15,000-c.m. capacity but can load and unload at both refrigerated and pressurized facilities. Refer to ISGINTT (2010) for further description of gas vessel types. Table 2-4 gives general dimensions and characteristics of selected LNG and LPG carriers. Such vessels are often dedicated to runs between two particular supply and receiving terminals. Their movements within harbors are controlled by very stringent safety requirements, and terminal facilities are of a very specialized nature. Further information on the early development of LNG vessels is presented by Thomas and Schwendtner (1971), and more contemporary LNG vessel technology and the largest carriers are described in SNAME (2010).

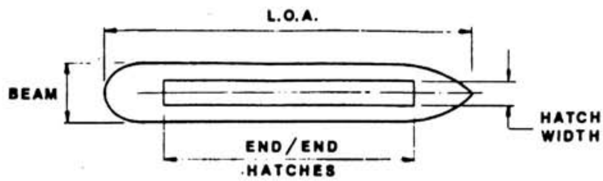
Table 2-4. Principal Dimensions of Selected LNG and LPG Carriers

Vessel Name/Type	LOA (ft)	Beam (ft)	Depth (ft)	Draft (ft)	DWT (l.t.)	Capacity (c.m.)
Mozah/Q-Max LNG	1,132.0	176.5	88.6	39.4	130,100	266,000
Al Oraiq/Q-Flex LNG	1,033.5	164.0	88.6	39.4	119,705	210,000
Abdelkader/LNG	977.7	150.9	87.9	39.0	—	177,000
Amali/LNG	932.5	142.4	85.3	37.7	73,600	147,000
Aquarius/LNG	936.0	143.5	82.0	36.0	63,600	125,000
Staffordshire/LNG	742.3	112.2	70.8	42.6	55,800	75,000
Sener/LNG	334.6 (LBP)	60.7	34.5	18.9	4,100	5,000
Pioneer Louise/LPG	747.8	120.0	72.2	37.6	55,843	77,500
Donau/LPG	600.2	98.4	56.1	38.9	32,339	30,207
Becquer/LPG	277.5	47.6	28.9	21.6	—	3,250

Combination Bulk Carriers are specially configured to carry both liquid and dry bulk cargoes; the most common type is the OBO (ore/bulk/oil). Other adaptations include ore/oil, bulk/oil, and ore/slurry/oil. These vessels are usually relatively large, typically in the 50-kDWT to 200-kDWT or more range. Their relative proportions are similar to those shown in Fig. 2-6 for tankers. These vessels have the flexibility required to engage in different trades and/or haul one cargo one way and a different cargo on the return voyage. Nearly all new vessels below 130 kDWT are OBOs with drafts generally less than 50 ft. Dorman (1966) provides detailed information on the early development of combination carrier characteristics.

Dry Bulk Carriers may be engaged in carrying a wide range of dry bulk cargoes, such as ore, coal, and grain, and may also be adapted for combination cargoes, such as bulk and containers, vehicles, wood chips, and other specialized bulk products. Dry bulk cargoes can vary greatly in nature, ranging in density from 10 c.f./l.t. for iron ore to 100 c.f./l.t. for certain grains, compared with the usual range of 36 c.f./l.t. to 48 c.f./l.t. for petroleum products. Cargo spaces are divided into *holds,* or individual compartments, to meet structural and stability requirements, as well as to restrain shifting of cargo, to allow different types of cargo to be carried, and for ballasting. Bulk carriers may be outfitted with their own cargo-handling gear, such as booms, cranes, or conveyor systems for self-unloading. Such vessels are referred to as "geared." Most dry bulk carriers are below 150 kDWT; general-purpose bulk carriers are dominant and in the 20- to 60-kDWT range. Panamax-size vessels are limited to about 80 kDWT. Larger bulk carriers of greater than 80 kDWT are often referred to as "Capesize" because they are too large to transit the Panama Canal. Handymax refers to vessels in the 35- to 50-kDWT range. The upper practical limit seems to have leveled off at around 350 kDWT, similar to tankers, although bulk carriers up to 400 kDWT have been built. Bulk carriers tend to have full hulls with long parallel midbodies similar to those of tankers and combination carriers. Bulk carriers differ from tankers in having deck hatches that typically cover from 50% to 75% of the midbody deck area. The size, number, and extent of hatch openings is of obvious importance in laying out berths and cargo-handling equipment. The typical principal dimensions and other characteristics of selected bulk carriers are given in Fig. 2-7; Jacobs (1983), Roseman et al. (1974), and Rowbotham (2014) provide discussions of bulk carrier evolution.

General Cargo Ships, sometimes referred to as "freighters," may carry a wide variety of cargoes, stowed in palletized, baled, crated, containerized, or other forms. *Break bulk cargo* refers to individually wrapped, packaged, crated, and other items that are stowed individually in the ship's hold, as compared with unitized cargoes that may be bundled or palletized together or containerized into larger units. General cargo ships may be multipurpose break bulk types or dedicated to specific commodities, such as refrigerated cargoes, lumber, or other commodities. Fully refrigerated ships are known as *reefers.* General cargo ships are typically self-unloading or "geared,"



D.W.T.	L.O.A.	BEAM	DEPTH	DRAFT	END/END HATCHES	HATCH WIDTH
40,000	207	27.5	16.5	11.5	135	13.0
60,000	235	32.0	18.0	12.8	155	14.0
150,000	305	43.5	25.0	18.0	220	20.0
200,000	327	52.0	27.0	19.1	230	22.0
250,000	344	54.0	28.5	21.0	245	24.0
350,000	390	61.0	34.5	25.5	300	25.0

NOTE: ALL DIMENSIONS IN METERS

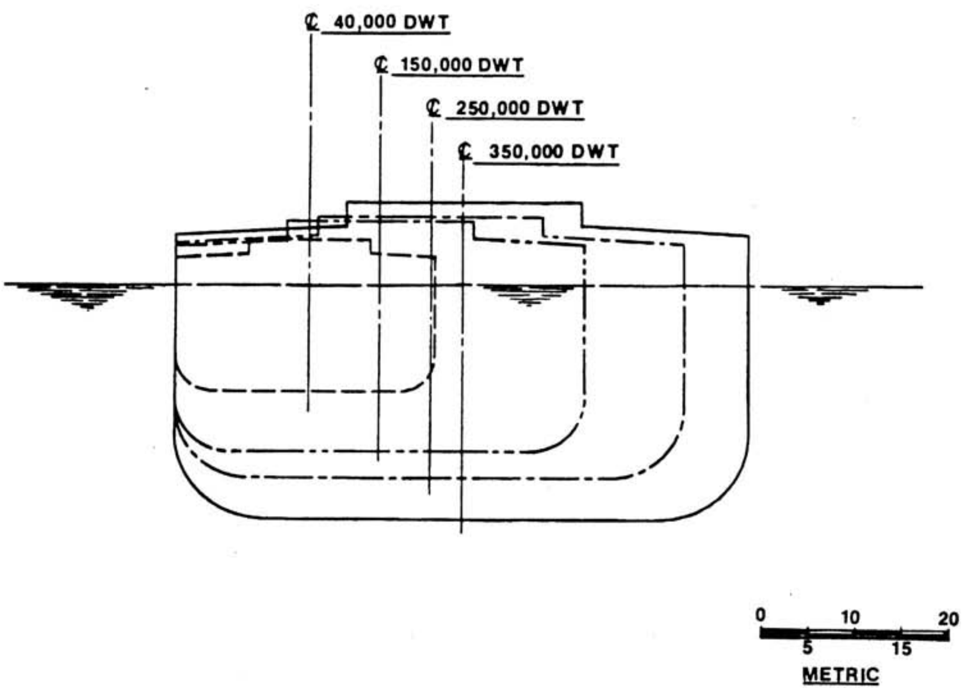


Fig. 2-7. Bulk carrier dimensions

Source: Courtesy of Soros Associates, Consulting Engineers equipped with their own cargo-handling gear ranging from approximately 20 tons to 100 tons (heavy lift) capacity. A typical midsized ship may have five or six holds. Vessels engaged in regular scheduled trade routes are often referred to as cargo liners, as compared with general-purpose tramp ships, in which the cargo and the destination vary with consignments. General cargo ships are typically within the 10- to 25-kDWT capacity range, and within 450- to 550-ft LBP with drafts on the order of 30 ft, allowing them to enter virtually all major U.S. ports. Small vessels may be used on shorter and coastal trade routes. Table 2-5 gives principal dimensions of representative general cargo vessels. See also Lamb (2003) and Whitehurst (1983).

Containerships are the current greyhounds of the sea, featuring faster turnaround times at port and a greater cubic capacity per DWT, as well as high speed en route. Containerships have become prominent over the years despite the fact that intermodal transport requires specialized port facilities and access to land transportation systems. Containership capacity is expressed in TEUs, or 20-ft equivalent units, referring to the number of equivalent 20-ft-long standard-size containers carried. Since the advent of containerized cargo in the mid-1950s, when mostly modified cargo ships and tankers were limited to less than 1,000 TEUs, dedicated containership capacity has grown to 19,000+ TEU capacity with several ships of more than 20,000 TEUs now on order. At the time of this writing, the largest containership in operation is the *MSC Oscar*, with 19,224-TEU capacity and 1,244-ft LOA, and a few vessels of a little more than 20,000 TEU are on order. The current upper limit set by shipbuilding facility capacity is around 24,000 TEU and 1,500-ft LOA. The earliest container-carrying ships were constrained to carry containers only on deck, but in the early to mid-1970s, as containerization caught on, dedicated ships with cellular holds capable of stacking containers in cells within the ship as well as on deck became the norm. The early development of containerships is addressed by Dorman and deKoff (1971), Boylston et al. (1974), and Sartor and Gibbon (1979). Fig. 2-8 illustrates the evolution of containership size and capacity, and Rodrigue (2013) provides more in-depth treatment of their historical development. Table 2-6 gives principal dimensions and other characteristics for selected vessels. Dimensions and weights of standard containers are given in Section 4.1. Note that the TEU

Table 2-5. Principal Dimensions of Selected General Cargo Ships

Class/Type	LOA (ft)	LBP (ft)	Beam (ft)	Depth (ft)	Draft (ft)	DWT (l.t.)	DT (l.t.)
Generic cargo liner	498.5	475.6	75.2	43.6	31.3	16,698	—
PD-159	504	470	74.0	43.0	28.3	11,850	17,900
Mariner class	563.7	528.5	76.0	35.5	29.9	12,910	21,093
Kennebec class	570	541	75.0	45.0	33.5	21,920	29,300
Merrimac class	592	550	86.0	54.8	34.0	15,900	26,400
Alaskan mail	605.0	582.6	82.0	46.5	35.1	22,208	31,995

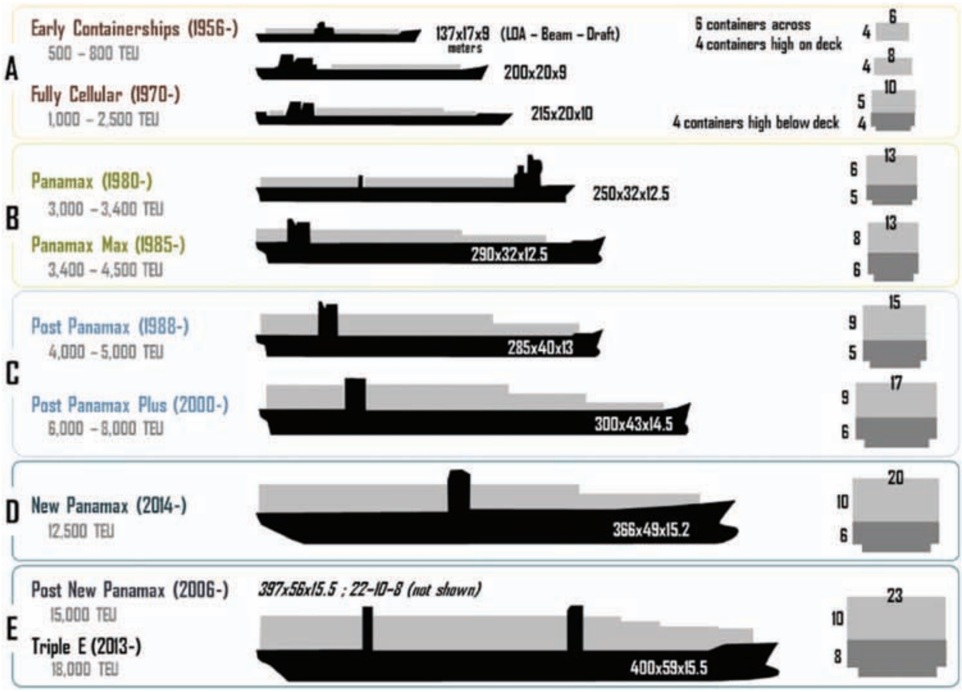


Fig. 2-8. Evolution of containership dimensions

Source: Rodrigue (2013); reproduced with permission of J. P. Rodrigue

Table 2-6. Principal Dimensions of Selected Containerships

Class or Identification	LOA (ft)	LBP (ft)	Beam (ft)	Depth (ft)	Draft (ft)	Capacity TEU	DT (l.t.)
CSCL Globe	1,312.3	—	192.3	100.1	47.6	19,000	155,200 DWT
MSC Camille	1,198.8	1,146.7	167.9	98.1	45.9	14,000	133,000
New Panamax (NPX)	1,199.0	1,146.4	158.0	97.8	48.9	12,600	119,800
MSC Heidi	1,089.2	1,040.6	141.7	80.4	42.7	8,400	89,150
NYK Lodestar	983.3	934.8	131.2	—	42.6	6,422	68,300 DWT
U.S. Lines	950	—	105.7	—	35	4,258	—
PD-161	941.0	862.0	105.0	64.0	32.0	2,294	44,600
C-10	902	—	129	—	41	4,300	53,648
President Lincoln	860	810	105.7	66	35	2,500	49,500
C8-5-85	813.3	769.0	90.0	53.0	33.0	1,708	42,350
PD-I60	758	706.0	101.0	55.0	28.5	1,672	32,500
C6-5-85	669.3	625.0	90.0	53.0	33.0	1,186	30,490

is not a precise measurement and certain vessel owner/operators may rate their ships' TEU capacity differently with regard to the number of containers that can be carried, assuming that they are loaded to a specific weight limit. The vast majority of shipboard containers are typically 40 ft long or 2 TEU per container (Section 4.1).

Ro/Ro, or roll-on/roll-off, vessels can accommodate virtually any over-the-road vehicle. They are essentially large oceangoing ferries originally envisioned as pure trailer ships and are typically equipped with stern, quarter, or side ramps and/or side loading cargo doors. The major drawback to such a system in competing with other means of cargo handling is its loss of cubic capacity because of the parking space required by vehicles. Ro/Ro vessels carry their own vehicle ramps and thus are self-unloading. They are most adaptable to shorter ocean routes, usually with a 1,000- to 2,000-mile range, because of the economics of loading/unloading containerized cargoes versus driving on and off. Most Ro/Ro vessels are less than about 20 kDWT capacity, although in the early 1980s, larger third-generation vessels (MLE 1984) approaching 40 kDWT and having combined container and Ro/Ro capability were introduced into the transoceanic trade routes. Table 2-7 gives principal characteristics of selected Ro/Ro vessels, and Argyriadus et al. (1979) and Taylor (1976) provide background on Ro/Ro design particulars. The designation Ro/Ro more broadly encompasses a variety of ship types, including trailer ships, train ships, and vehicle carriers dedicated to specific rolling stock and passenger/vehicle ferries known as "Ro/Pax" vessels. Ro/Ro vessels that also carry general cargo may have their capacities expressed in car equivalent units (CEUs).

Vehicle Carriers are distinguished from the more general Ro/Ro-type vessels in that they carry only the vehicle (usually autos going to market) and not loaded

Table 2-7. Principal Dimensions of Selected Large Roll On/Roll Off Vessels

Name/Type	LOA (ft)	LBP (ft)	Beam (ft)	Depth (ft)	Draft (ft)	DT (l.t.)	Capacity (as Noted)
Barber Blue Sea	859.4	808.2	105.8	68.9	38.4	—	43,000 DWT (630 cars + 2,464 TEU)
ACL G-3	820.2	761.2	105.8	66.4	36.1	—	37,000 DWT (600 cars + 2,130 TEU)
Generic Ro/Ro[a]	684.0	640.0	102.0	69.5	34.0	33,765	18,989 DWT
Car carrier	639.6	—	105.0	53.8	36.0	—	20,600 DWT (5,500 cars or 921 TEU)
Atlantic Saga	—	600.4	90.0	63.3	29.5	27,531	15,725 DWT (1,200 cars or 500 TEU)
Australian Enterprise	—	551.2	82.0	53.8	29.5	22,173	14,082 DWT
USNS Comet	—	465.0	78.0	48.9	27.1	18,286	10,111 DWT

[a]A representative contemporary design with slewing quarter ramp and container carrying capability.

trailers with variable cargoes. Vehicle carriers have very large volumetric capacities and are easy to identify with their typically very large, wall-sided superstructures. Vehicle carriers capable of carrying on the order of 5,000 automobiles, corresponding to approximately 15 kDWT, are in service. Also, vehicles often are carried as an alternate cargo on combination carriers. Because of their relatively low-density cargoes, Ro/Ros and vehicle carriers typically have higher freeboards, lower drafts, and slightly larger overall dimensions than general cargo vessels of the same DWT capacity.

Barge Carriers in the United States have primarily been built according to one of two patented systems: the LASH (lighter-aboard-ship) system and SEABEE the system, which is now out of service. The LASH vessels have square, open sterns with gantry cranes (LASH) for lifting barges aboard. The standard LASH barges are relatively small at 61.5 × 31 × 13-ft depth and 9.5-ft draft and are brought to the vessel's stern by tugs or towboats and lifted aboard. SEABEE barges are 97.5 × 35 × 10.5-ft draft. LASH vessels of up to 893-ft LOA × 100-ft beam and 31-ft draft capable of carrying up to 89 barges have been built. Alternatively, on some barge-carrying vessels, such as the now defunct European BaCo system, the barges may be floated aboard by partially submerging the vessel. Barge carrier systems may be competitive where river or canal systems exist at each terminus, especially in underdeveloped areas, and where a limited number of port calls are involved, but they are not widely used currently and appear to be falling into disuse. For an overview of the design and history of the LASH and SEABEE ships, refer to Benford and Fox (1993).

Ferries, which are used principally to transport highway traffic vehicles and passengers, typically form a bridge or link between overland roadways. They vary in size from small river-crossing and harbor commuter ferries carrying fewer than a dozen automobiles, to large oceangoing ferries carrying more than 200 vehicles on overnight runs. Such ferries may also be classed as Ro/Ro type (Ro/Pax) vessels, as opposed to passenger-only-type ferries. Double-ended ferries, with propellers at both ends, are frequently used on shorter commuter runs, whereas single-ended ferries are more economical and are preferred for exposed water crossings. Ferryboats typically have broad beams to maximize the number of cars carried and provide stability for the relatively high center of gravity of the cargo and superstructures. Ferryboats also typically have high B/D ratios, in the 4 to 5 range and higher for side loaders. They also are typically fine-lined below the waterline to reduce wave-making resistance, with a C_B in the 0.4 to 0.5 range. High-speed catamaran-type vessels have somewhat revolutionized commuter service and are now in common use; they are being built to capacities similar to those of many larger mono-hull-type vessels. Table 2-8 presents some representative ferry characteristics.

Cruise Ships originated with the development of the cruise ship industry in the early 1970s and have doubled in size and capacity every 10 years, representing six generations (Levander 2011b). Fig. 2-9 illustrates cruise ship development trends. The first generation represented by the *Song of Norway* was 552-ft LOA and 18,400 GT and carried 750 passengers, whereas the sixth generation and current record holder,

Table 2-8. Principal Dimensions of Selected Ferries

Type	LOA (ft)	LWL (ft)	BOA (ft)	BWL (ft)	Depth (ft)	Draft (ft)	DT (l.t.)	Capacity (as Noted)
Baltic Sea Ferry (Ro/Pax)	545.3	—	93.2	—	—	31.3	—	1,440 passengers/283 autos
Alaskan Ferry (Ro/Pax)	418.0	385.0	85.1	72.0	24.0	16.0	6,700	1,000 max. passengers/326 autos
Nova Star (Ro/Pax)	528.2	—	84.0	—	—	19.9	27,744 GT	1,215/1,575 lane meters
Tokitae (de) Olympic Class	362.0	—	83.3	—	—	18.0	4,384	1,500/144
Puget Sound (de)	328	317.5	78.5	62.8	23.0	15.5	2,930	300 max. passengers/100 autos
Staten Island (de)	294.0	285.0	60.8	50.9	20.6	13.5	2,360	300 max. passengers/175 autos
"Jumbo Cat" (high-speed catamaran)	244.8	—	63.2	—	—	10.2	—	449 passengers/121 autos
Nantucket Ferry	239.5	—	60.1	—	17	11	1,565	980 max. passengers/50 CEU
Island Home (de) (WHMVNSA)	254.7	238.7	64.0	—	17.5	10.8	1,928	1,200/76
Wight Light (de)	203.4	—	52.5	—	—	7.5	1,495	65 CEU
Anna C	195	—	40	—	—	9.5	98	1,300 passengers/40 autos
Casco Bay Ferry	133.0	—	36.3	—	—	7.8	255	10 autos/passengers not available

Notes: The label (de) denotes double ender. CEU means car equivalent units.

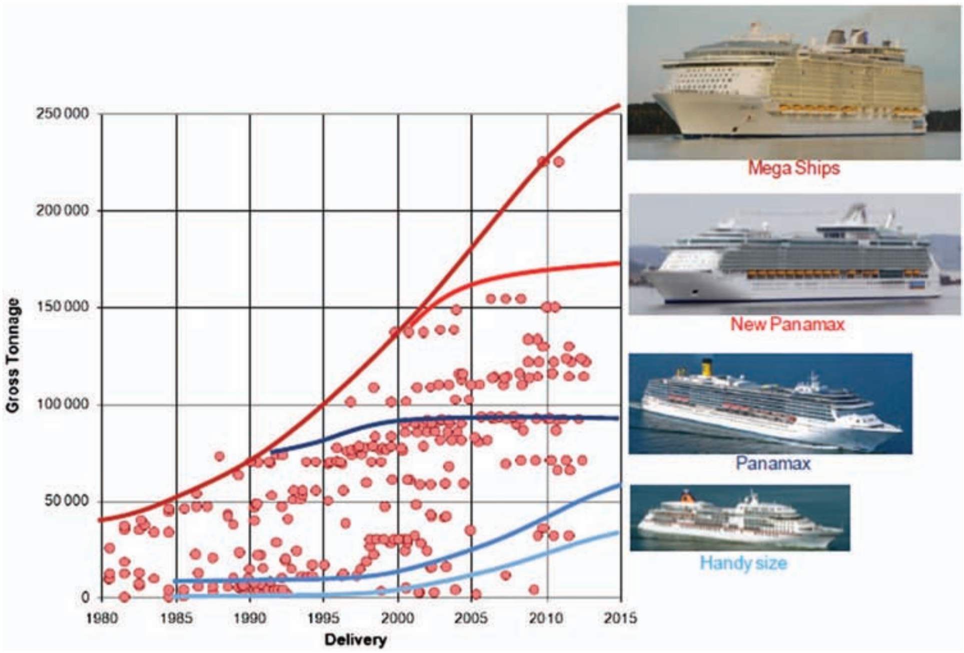


Fig. 2-9. Cruise ship development trends

Source: Levander (2011b); reproduced with permission from The Society of Naval Architects and Marine Engineers

represented by the *Oasis of the Seas* completed in 2009, is 1,186-ft LOA and 225,282 GT and carries approximately 6,300 passengers. Cruise ships represented somewhat of a resurgence of the once glamorous passenger ships, often referred to as ocean liners, of the early to mid-twentieth century that went into decline since their pre– and post–World War II heyday. The great transatlantic liners once were among the fastest and most majestic vessels ever built, such as the S.S. *United States,* which still retains the fastest commercial vessel transatlantic crossing record but has long since been retired. The Cunard Lines *Queen Mary II* was once the largest and fastest vessel of its type ever built, with an LOA of 1,132 ft, beam of 134.5 ft, and height of 236 ft. Today's cruise ships are not so much involved with transporting people as with providing onboard vacations in the form of cruises to no specific destination or to local ports of interest. Contemporary cruise ships are typically fitted with bow and stern thrusters or pod drives that allow the vessels to berth without tug assistance and turn around virtually within their own length. Principal dimensions for selected cruise ships are given in Table 2-9.

The domestic cruise ship industry and its vessels have been well described by Leeper and Boylston (1987). The short-range mini-cruise business has brought about an essentially new class of small passenger vessel (Cashman 1985). Adventure cruises are now offered to more remote sites as well. Smaller passenger vessels

Table 2-9. Principal Dimensions of Selected Passenger Cruise Ships

Name	LOA (ft)	LBP (ft)	Beam (ft)	Draft (ft)	DT (l.t.)	GRT	Capacity (Passengers and Crew)
Oasis of the Seas	1,213.9	—	154.2	30.5	—	225,000	6,300/—
Quantum of the Seas	1,141.0	—	135.8	27.9	—	168,666	2,094/849 cabins
Freedom of the Seas	1,112.0	995.0	126.6	27.9	—	158,000	3,600/1,400
Britannia	1,082.1	—	125.9	27.2	—	143,730	4,324/1,376
Queen Elizabeth II	963.0	887.1	105.0	32.5	48,886	—	—
Michelangelo	904.8	800.4	101.7	30.6	41,328	—	—
Sovereign of the Seas	874.1	774.1	105.6	24.8	—	74,000	2,276/750
Ocean Princess	857.2	726.4	105.6	26.6	39,997	77,499	2,272/870
Royal Princess	761.2	—	95.8	25.6	—	40,000	1,200/—
Tropicale	669.3	580.7	86.3	23.0	—	36,600	1,396/491
Sea Venture	553.7	475.7	80.7	24.6	15,685	19,903	650/301
Cunard Adventurer	448.0	413.0	70.5	19.2	9,774	14,555	750/300
Grand Palais (casino riverboat)	360	—	99	—	—	—	2,000/500
Sea Goddess	343.8	—	47.9	14.0	—	4,000	120/70
Majestic Explorer	152	—	31	8	—	99.7	44 passenger staterooms

without sleeping accommodations are engaged in sightseeing and harbor tours. Principal dimensions and data for selected passenger-type vessels are included in Table 2-9.

Barges are a common sight in most ports, used to transport a wide variety of cargoes, some of which call for specially adapted hulls. The most common general types are deck, hopper (both open and covered), and tank barges. They may also be dedicated to specific cargoes, such as cement-bulk, rail cars (sometimes referred to as car floats), chemical, wood-chip, logs, and a variety of other special purposes (Ward 1980). Most barges have flat, raked, spoon-shaped, or ship-shaped bows and a straight flat transom with some bottom slope and skegs for directional stability. The length of parallel midbody is typically 80% or more of LOA. Barge sizes range up to more than 900 ft LOA and 50 kDWT. A common size for lower inland waterways deck, hopper, and tank river barges is 195 × 35 × 9 ft loaded draft with a capacity of 1,700 short tons. River barges are often rafted in integrated rows called "strings". Additional information on inland waterway barges and towboats characteristics can be found in AASHTO (2008) and Hupp (1977). A barge's proportions and dimensional ratios may vary greatly, depending upon its purpose, although values of C_B typically remain at around 0.95 or greater. Barges also may be used as platforms for floating equipment, especially lifting equipment, such as stiff-leg derricks or revolving cranes. Details of barge design and construction can be found in the ABS rules for barges (ABS 2015b).

An important adaptation of barges in marine transportation is the integrated tug/barge (ITB) system. In this system, the tug and barge are rigidly linked to act as a single vessel with the tug's bow secured inside a notch in the barge's stern. The bow of the barge is ship shaped, rounded, or spoon shaped, and when linked together the ITB combination somewhat resembles a bulk carrier vessel in its proportions. Most ITBs are engaged in dry bulk or liquid bulk transport. This system of transport has certain advantages when the loading time is long compared to the transit time, the cargo movement is intermittent or seasonal, and the cargo is stockpiled or warehoused. Further information on ITBs can be found in Seymour and Kossa (1981) and Giblon and Tapscott (1973). A further refinement of the ITB system is the articulated tug/barge (ATB or AT/B) system, wherein the hinged connection between tug and barge allows relative movements between their hulls that offer certain seakeeping advantages as well as tugs that can be designed for multipurpose use, unlike ITB tugs, which are pretty much dedicated to a specific barge type. Many ATBs are dedicated to the coastwise distribution of liquid petroleum products, although ATBs are also used in transoceanic routes. The Crowley 750 Class ATB is among the largest currently in service and has a rated capacity of 327,000 bbl and an LOA of 600 ft (barge alone) and 674 ft integrated with tug. The development and state of the art of the ATB is described by Hill (n.d.), and a case study of ATB design is presented by Wolff (2003).

Workboats and Service Vessels include tugboats, towboats, offshore supply vessels, firefighting vessels, pilot boats, and various other utility craft. Tugboats and towboats are distinguished from one another in that tugs may either pull (tow) or push, whereas towboats normally push. Towboats are most common on inland waterways, where they transport rafted barge tows. Towboats normally have heavy bow knees for engaging their tow, their superstructures usually extend almost the full length of their hulls, and often they have elevated pilot houses so that operators can see over their cargo. Tugs designed primarily for towing generally have lower superstructures that terminate well forward of their sterns so that the tow rope pull acts close to the vessel's turning center, allowing better control of the tow. Tugs usually are engaged in assisting larger vessels in berthing and escort service, general towing, salvage, and rescue and generally can be categorized by size as harbor, coastal, and oceangoing. *Tractor* tugs are especially useful for berthing ships and certain construction applications as they have fully azimuthing propulsion systems, which allow full thrust in any direction relative to the tug's heading. The earliest tractor tugs were equipped with Voith Schneider vertical axis cycloidal propellers (VSPs) mounted near the tug's midship. Many contemporary harbor tugs are equipped with *Z*-drive units consisting of shrouded propellers that are mounted on pods capable of being rotated 360 degrees, thus eliminating the need for an independent rudder and allowing fully azimuthing thrust similar to cycloidal drives. *Z*-drives are often mounted further aft than cycloidal drives, thus losing some maneuverability but retaining some of the benefits of traditional stern propellers for towing service.

Because their primary use is in moving loads, tugs and towboats generally are referred to in terms of their pulling power or engine (brake) horsepower (bhp), or often the power delivered to the shaft, or shaft horsepower (shp). Harbor tugs usually range from 50 to 100 ft LOA and 200 to 1,000 shp, coastal tugs from 100 to 120 ft LOA and 1,000 to more than 3,000 shp, and oceangoing tugs from 150 to more than 250 ft LOA and up to 23,000 shp. Tractor tugs are most often in the range of 80 to 100-plus ft and 3,000 to 5,000 bhp, although a few tugs longer than 150 ft LOA and up to 10,000 bhp have been built. Traditional harbor and coastal tugs typically have drafts within the range of 8 ft to generally less than 15 ft and are characterized by a relatively low L/B in the 3 to 4 range and a moderate to high B/D in the 2.2 to 3.2 range. Traditional harbor and coastal tugs typically have raked keels so that it is important to verify that the reported draft is the maximum aft, navigational, versus the mean draft used to determine coefficients. They have moderately fine lines below the waterline, with C_B values in the range of 0.52 to 0.58. Displacements typically range from 100 l.t. to less than 1,000 l.t. An important result of the tug's power and propulsion system is its towline pull, or bollard pull, which is usually given at zero speed (or 100% propeller slip). The bollard pull of a tug with a diesel electric power plant can be roughly estimated at 25 lb per bhp for conventional propellers and 30 to 35 lb per bhp for screw tugs equipped with Kort nozzles. The Kort nozzle is a ducted propeller shrouded by a nonrotating nozzle that improves the propeller efficiency but still requires a conventional separate rudder. See Section 6.11 for additional information on tug bollard pull capacities and use of tugs in port operations.

The largest tugs generally used for docking vessels usually are within the 3,000- to 4,000-bhb range, and must operate cautiously to prevent damage to the berthing vessel's hull plating. The tug's characteristics that are most important to docking operations include its bollard pull capacity and effective use of its power, maneuverability, and stability while thrusting against a vessel. Contemporary harbor tugs have bollard pulls in the 50- to 80-ton range, which may equal or exceed the capacity of a ship's mooring fixtures, and thus tugs of this capacity may represent a practical limit for harbor operations. Criteria for matching tugboat specifications to vessel berthing requirements can be found in Allen and Woodward (1988) and NPC (1977). Treatment of the general design of tugs and typical dimensions and characteristics can be found in Lamb (2003) and Spaulding (1973).

Another prominent workboat is the offshore supply vessel (OSV), with a limited superstructure well forward and plenty of large open deck space aft. Large OSVs equipped with towing machinery, which are called anchor handling tugs, are used to tow and deploy offshore rigs and moorings. OSVs have evolved from the early supply boats of generally less than 150 ft LOA and 1,000 shp to more than 200 ft LOA and up to 10,000 shp for contemporary offshore tug and supply vessels. Many of the earlier generation of OSVs have found their way into various commercial transportation uses and services outside of the offshore oil industry. Like traditional harbor tugs, they are characterized by a generally low L/B but an even higher B/D, in the range of 3 to 4, reflecting their relatively shallow draft and wide-open deck capacity. They also have slightly higher values of C_B, in the range of 0.62 to 0.75 when fully loaded. The larger contemporary OSVs tend to have more moderate proportions, with an L/B in the 4 to 5 range and a B/D in the 2.5 to 3.0 range. A detailed description and dimensional data for many of these vessels can be found in Mok and Hill (1970), Raj and White (1979), and Patten (1983). The offshore industry uses a wide variety of specialized service vessel types somewhat similar to OSVs, such as anchor handling tugs, well stimulation vessels, seismic survey vessels, crew boats, and others that are too varied and numerous to cover herein.

Numerous specialized vessel types fall under the industrial and miscellaneous service vessel categories. Fishing vessels are perhaps the most common of vessel types worldwide, ranging in size from the smallest skiff to tuna catchers of 250 ft LOA or more to factory ships or mother ships of up to around 30,000 DWT. Their sizes and arrangements vary greatly with the type of fishing carried out. Most fishing vessels, however, have in common relatively high displacement ratios or cubic coefficients because of their need to maximize catch capacity. Descriptions of fishing vessels and their gear can be found in Lamb (2003), Cole (1988), and Blair and Ansel (1968).

Another fairly common industrial-type vessel is the oceanographic research vessel. These vessels may be specially built for their given purpose or converted from some other vessel type, so their characteristics vary widely, as described by Daidola and Griffin (1986). Information on U.S. university research vessels can be obtained through the University-National Oceanographic Laboratory System (UNOLS) website (Appendix 3). Other important industrial vessel types include dredgers, pipe-laying and ocean construction platforms, cable ships, ocean drilling and mining vessels, and various floating plants, such as incinerator vessels. Dredges are a relatively common site of obvious importance to port and harbor maintenance. Dredges can be broadly classified as mechanical types, including bucket, dipper, and ladder types and a wide variety of hydraulic types that not only excavate material but also dispose of it directly either by pumping it through floating pipelines or storing it on board in hoppers for release at designated spoil disposal sites. The self-propelled trailing-suction hopper dredge is among the most versatile and common types operating worldwide today. See Herbich (1992) for an in-depth description of dredges and dredging operations and Clarkson (2010) for information on the world's dredge fleet. *Liftboats* are essentially self-propelled jack-up barges used primarily in offshore and deepwater port construction and pipe-laying operations. Fig. 8-1 shows a representative-type liftboat engaged in offshore subsurface investigations. Some such vessels can operate in up to 200-ft water depth. For a more detailed description of floating construction equipment and operations, see Gerwick (2000).

Other miscellaneous vessels of interest include certain advanced vehicle types, such as surface effect ships (SES), hydrofoils, and various catamaran (twin-hull) and SWATH (small waterplane area twin-hull) configurations that are increasing in use, particularly for rapid passenger transport (Kennell 1998). Information on yachts and recreational small craft can be found in the general small-craft references cited in Section 3.3. "Mega-yachts," with more than 100 ft and up to around 600 ft extreme LOA-like small passenger ships with many amenities and very small passenger capacity, are described in PIANC (2013). General dimensions of the world's top 100 largest yachts are published annually by *Yachts International* magazine (Appendix 2).

Military Vessels include a wide variety of vessel types and sizes, ranging from the smallest patrol craft to the largest aircraft carriers. U.S. Navy vessels can be broadly classified as *combatant* ships and craft that are armed and intended for warfare, *auxiliary* ships intended as support ships for combatants and other naval activities, and *yard and service* craft. Ships and large craft are assigned hull numbers that follow a vessel-type letter designation. The letter *T* before the vessel type designation indicates a Military Sealift Command (MSC) vessel that is typically a commercial cargo ship that has been appropriated or chartered by the Navy for supply and/or advanced positioning purposes. U.S. Maritime Administration (MARAD) owns and maintains a ready reserve fleet (RRF) of cargo and support vessels (USDOT 2014) that may be operated by the MSC in support of U.S. Army and U.S. Marine operations. The RRF also includes Maritime Academy training ships. Characteristics and dimensions of U.S. Navy vessels can be found in the NAVFAC *Ships Characteristics Database* (SCDB), which can be accessed on line through the Whole Building Design Guide (WBDG) website (see Appendix 3, Polmar (2013), and NAVFAC (1986) for printed particulars of the U.S. Navy fleet). Information on the world's combat fleets can be found in Jackson and Crawford (2004).

The U.S. Coast Guard (USCG) operates a wide variety of vessel types and sizes, all of which are generally referred to as "cutters" when they are more than 65-ft LOA.

Medium-endurance (WMEC) and high-endurance (WHEC) cutters are among the larger vessels generally involved in national security and law enforcement; smaller patrol craft are involved in search and rescue (SAR) missions as well. The USCG also operates icebreakers, buoy tenders, and various tugs and numerous small craft. For information on the USCG fleet, see Polmar (2013). The U.S Army also operates its own fleet of vessels, including logistics and landing craft and coastal tugs generally involved in harbor maintenance activities, and the U.S. Army Corps of Engineers maintains a small fleet of dredgers, including hydraulic pipeline, hopper, and bucket dredges. The National Oceanographic and Atmospheric Administration (NOAA) operates a fleet of ocean research and survey vessels.

Additional tabulated and graphical data on various vessel types can be found in the cited literature and the general references on port and harbor engineering given in Chapter 1. PIANC (2002) presents tables of generic vessel dimensions and characteristics, including wind areas for various vessel types and within-specified-confidence limits useful for preliminary design and where actual design vessels are uncertain. Several marine engineering and shipping industry professional journals and trade magazines regularly publish detailed data on vessels of all types (Appendix 2), and many other sources are available on the Internet (Appendix 3).

In summary, it is essential that the harbor engineer seek out specific vessel data for those vessels that will use and/or are likely to visit the facility being designed. The dimensional data presented here on representative vessels of the types most commonly encountered by the port engineer are included for general information and instructional purposes. However, it is most important that the port and harbor engineer understand a ship's basic hull form and proportions as they relate to the vessel's mission and berthing requirements. Subsequent chapters illustrate how a vessel's shape and proportions relate to berthing and mooring loads and how its hull structure relates to berthing and dry docking design considerations.

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CHAPTER 3

General Design Considerations

The establishment of definitive design criteria is of primary importance in the design of any structure, and experienced judgment is most important in this area. Design criteria follow from evaluation of various functional and operational requirements as well as site and environmental conditions and are eventually translated into design loadings and serviceability requirements, as described in subsequent chapters. Judicious selection of appropriate construction materials is critical to the long-term performance of the structure.

This chapter begins with a review of general design considerations affecting the overall planning, design, and construction of a marine facility. In the following sections, site selection and layout criteria are briefly reviewed and specific facility type requirements are discussed. Environmental conditions and means of describing them for design purposes are reviewed, and a final section on materials gives a general description of marine construction materials and criteria for their selection.

3.1 Design Criteria and General Considerations

Design criteria for marine structures should be established after careful consideration of various operational, functional, and navigational requirements, environmental and site conditions, and physical and regulatory constraints, as summarized in Table 3-1. Most of the items in the table are self-explanatory, and they are often dictated by the owner/operators and their requirements, existing conditions within the port, regulatory constraints of government agencies, and, possibly, port security issues. Design criteria should be summarized in a "basis of design" document prepared at the outset of a given project. Structural design criteria in particular should include the following items:

- Overall dimensions and configuration;
- Deck elevation;
- Water depth alongside, plus dredge and scour allowance;
- Deck, cargo, and equipment loads (Chapter 4);
- Berthing loads—design vessels, speed, and approach angles (Chapter 5);

Table 3-1. General Design Considerations for Marine Facilities

Site Conditions

- Topography
- Bathymetry: soundings
- Subsurface data: geologic history, soil properties, depth to rock, etc.
- Exposure to wind and waves
- Seismicity

Environmental Conditions

- Meteorology: normal and extreme, wind, rainfall, temperature
- Oceanography: normal and extreme wave, tide, current, ice, water chemistry, seiche or harbor surge, etc.
- Frequency and probability of storm conditions

Operational Considerations

- Vessel data, sizes, types, frequency, berth occupancy time, loading and servicing requirements
- Vehicle data, sizes, types, capacities, operating dimensions (turning radii, etc.)
- Trackage, cranes, loaders, railroad, capacity, weights, windage, gauge, speed, reach and swing, etc.
- Special equipment, mooring hardware, capstans, loading arms, product lines, etc.
- Services and utilities, shore connections, fire protection and safety equipment, lighting and security, electrical power, piping
- Cargo storage area

Functional Considerations

- Dredging, scour and siltation, propeller wash
- Vessel traffic and traffic control systems (VTS)
- Land-side access, remoteness, roadways, airports, etc.
- Maintenance practices: cathodic protection, damage repair, etc.

Navigational Considerations

- Channel depths and widths
- Vessel approach conditions
- Navaids
- Availability of tugs

Constraints

- Harbor and pier-head lines
- Regulatory: water quality standards, oily ballast, dredge disposal, fill, etc.
- Permits and licensing
- Availability of materials and equipment
- Existing facility: changed usage or upgrading limitation

- Mooring loads—design vessels, wind, and wave and current action (Chapter 6);
- Design life and durability requirements;
- Materials and construction methods;
- Allowable stresses and factors of safety (Chapter 7); and
- Applicable codes and standards (Chapter 7).

The first six items in the list are a direct function of the design vessel or vessels and their cargo-handling requirements, the first three involving spatial requirements and the next three, loads and forces. Load evaluation is an area in which marine structures differ most from traditional civil engineering structures, so individual chapters of this book have been devoted to direct structural loadings and berthing and mooring loads. These loads in turn depend upon the establishment of other design criteria, such as design storm conditions, and vessel operational limits, such as berthing velocities and berth approachability and occupancy criteria, all of which are probabilistic in nature and are discussed in subsequent chapters.

Fig. 3-1 illustrates various sources of loadings and design considerations for a typical pier structure, which is discussed in turn in following sections. Because of the exposure of marine structures to the natural elements and the probabilistic nature of vessel effects and accidental overloads, the design of marine structures traditionally has been characterized by a modest degree of conservatism. However, in more recent years, with the application of computers, a better understanding of environmental forces, a probabilistic approach to design, and better quality control of material structures have been built to the economic limit of the established design criteria.

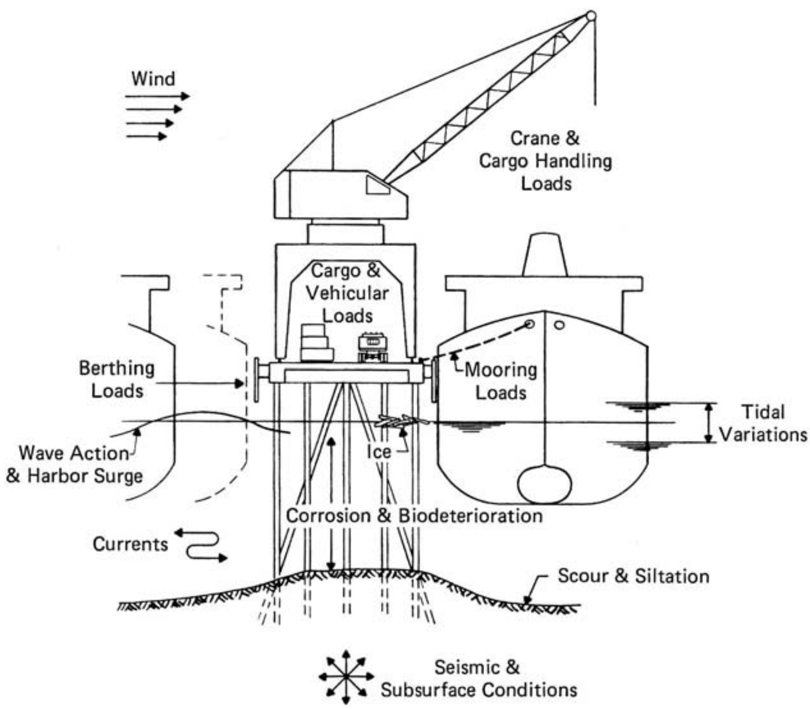


Fig. 3-1. Generalized loads and environmental factors affecting pier design

Selection of an appropriate "design life" as a basis of design for a port structure has an important influence on the determination of other design criteria, including the following:

- Consideration of long-term, time-dependent processes, such as corrosion, biodeterioration, fatigue, and soil settlement;
- Determination of probability levels for design storm, extreme water levels, earthquakes, and other natural hazards; and
- Determination of economic feasibility and cost analysis and consideration of possible future developments.

Major port facilities and most public works facilities usually have economic design lives on the order of 50 to 80 years. U.S. Navy and many commercial facilities have nominal design lives on the order of 25 years, with practical service lives of up to 50 years or more with periodic maintenance or upgrading. In fact, many U.S. facilities of post–World War II vintage have been or are being rehabilitated, upgraded, or converted to a new use as an economic alternative to new construction or as an environmental alternative to usurping precious wetlands (see Section 11.1). The term "service life" is specific to a structural component or material and is defined as the period of time after installation during which all the properties exceed the minimum acceptable values when routinely maintained. Three types of service life can be defined (ACI 2000):

1. Technical service life is the time in service until a defined unacceptable state or safety level or failure is reached.
2. Functional service life is the time in service until the structure no longer fulfills its functional requirements or becomes obsolete because of a change in the requirements.
3. Economic service life is the time in service until replacement of the structure or its parts becomes economically more advantageous than keeping it in service.

Probability and risk considerations are important in the determination of design loads and operating conditions consistent with economic realities. Structures for which governing design loads are associated with natural events such as storm winds and tides usually are designed for an event associated with a statistical return period of 50 to 100 years or more. The statistical return period (T_R), the encounter probability (E_p), and the design life or interval (n) over which the likelihood of encounter is being evaluated are related as follows:

$$E_p = 1 - (1 - 1/T_R)^n \tag{3-1}$$

The T_R and n normally are measured in years and E_p in percent. Therefore, in any given year ($n = 1$), there is a 1% probability of occurrence of the 100-year event.

Table 3-2. Encounter Probability versus Design Life and Return Period

Design Life (years)	Return Period (years)						
	5	10	25	50	100	200	500
1	0.200	0.100	0.040	0.020	0.010	0.005	0.002
5	0.672	0.410	0.185	0.096	0.049	0.025	0.010
10	0.893	0.651	0.335	0.183	0.096	0.049	0.020
25	0.996	0.928	0.640	0.397	0.222	0.118	0.049
50	0.999+	0.955	0.870	0.636	0.395	0.222	0.095

A structure designed for the 100-year event stands a 22% chance of encountering such an event over a 25-year design life and a 64% probability of encountering the 25-year event (Table 3-2).

The return period of the event causing some predictable level of damage is often traded off in cost-optimization studies, allowing for the cost of certain maintenance and repairs to be carried out over time. This type of cost analysis commonly is carried out in the design of breakwaters and shore protection structures, but it may not necessarily result in very great cost savings for most pier and wharf structures, except perhaps in areas of high seismic risk. Other risk criteria used to evaluate cost include frequency distribution of total damage, mean total damage, and probability of zero-damage criteria (Borgman 1963). More elaborate techniques are often applied to the reliability and risk analysis of offshore platform structures, as described by Stahl (1986).

Virtually all risk analysis depends upon the determination of the return period of some major storm event. Return periods most often are determined by extrapolating data from relatively short periods of record to long-term probability distributions, using any of several extreme-value probability laws. Alternatively, the prediction of long-term extreme values of random events may be made from statistical models of the actual physical processes involved (Borgman 1975). PIANC (2012a) presents a general methodology of analyzing wind and wave (Hydro/Meteo) statistical data and their effect on ship and port operations. In terms of exceedance statistics, we are often more interested in the joint probability of occurrence of more than one environmental parameter, such as the highest wind speed from a given direction combined with extreme tide height, which in turn may result in the most damaging wave conditions. In this case, a worst-case design storm could be developed based upon the coincidence of meteorological conditions and astronomic tide data.

Regulatory Agencies

Of the many government and regulatory agencies that may be involved with planning, permitting, operating, and/or design requirements, the following are of importance in most U.S. port projects.

The U.S. Army Corps of Engineers (USACE) is responsible for dredging and maintaining navigable waterways and for granting permits for construction thereon, including inland waterways, under the Rivers and Harbors Act of 1899. The USACE has additional authority for control of dredged and fill material and its transport under the Clean Water Act and the Marine Protection, Research, and Sanctuaries Act of 1972.

The U.S. Coast Guard (USCG), in addition to enforcing maritime and pollution laws, maintains and controls aids to navigation (ATONs), performs search and rescue (SAR), and is responsible for icebreaking operations and the reporting of ice conditions and for port security. Port security is an important issue and may require guard towers, controlled vehicular and pedestrian access points, security lighting, "giant voice" megaphones in addition to on water and landside surveillance. Such features and U.S. government requirements are described in UFC 4-025-01 (DOD 2012).

The Environmental Protection Agency (EPA) oversees or controls development with potentially adverse environmental effects. Environmental protection usually is administered at the state level, and the number of agencies involved and modus operandi vary from state to state.

The Coastal Zone Management Act (CZMA) is an act passed by the U.S. Congress in 1972 to encourage coastal states to develop coastal zone management plans for the protection and improvement of coastal areas, and most states have adopted such federally funded plans, which are administered locally. On the local level, a city or town conservation commission and a harbor commission and harbormaster, fire marshal, public safety officials, and possibly other groups may review and approve plans for proposed construction. A few of the busiest ports and waterways are monitored by vessel traffic control systems (VTS) that could influence site selection and layout. Geographic information systems (GIS) have found applications in port operations and security as well (Wright and Yoon 2007).

The Maritime Transportation Security Act of 2002 (MTSA) and implementation of the International Maritime Organization's (IMO) International Ship and Port Security Code (ISPS) applies to all U.S. facilities that handle certain dangerous cargoes (CDCs), vessels with more than 150 passengers, and all international traffic. Port facilities engineers need to be aware of these regulations and their possible effect on future designs.

Planning Considerations

An important general consideration in the design of marine facilities is the possibility of future events that may have a profound effect on a proposed structure's function or integrity, such as future dredging, channel widening, increased vessel traffic, changes in vehicular and/or intermodal access, and construction on adjacent sites. The structure's design should also consider the possibility of future expansion and upgrading (PIANC 1987). Lifecycle management (LCM) (PIANC 1998a, 2008a) considers long-term overall management of planning and cost control, including construction, operations, maintenance, safety, and other issues over the life of the port or structure. Port planning in general is outside the scope of this book; however, the designer of marine facilities should be familiar with important basics. Port planning has been addressed by important international organizations, such as UNCTAD (1985), IAPH (2001), and more recently the PIANC Report No. 158, *Masterplans for the Development of Existing Ports* (PIANC 2014a), provides an overview of port planning issues. Textbook treatment of port planning can be found in the general references listed in Section 1.5.

3.2 Site Selection and Layout

The relative exposure of a site to wind and sea has a major effect on its structural design features. In the past, the majority of piers and wharves were situated in relatively well protected natural harbors and within rivers and estuaries, where wave action did not have a significant effect on terminal structures' layout and design. More recently, however, with the advent of larger and deeper draft vessels, marine terminals have been constructed in increasingly exposed locations, such as large natural bays or artificial harbors, with perhaps partial protection from prevailing seas afforded by a breakwater or more distant land masses. Offshore terminals consisting of "sea-islands" or mooring buoy arrangements with a pipeline or conveyor system link to shore may be constructed in essentially fully exposed locations. Depending upon the frequency and duration of storms and limiting sea states for operations, such terminals may have limited or seasonal berth occupancy times. Major port and marine terminal projects may require extensive field monitoring and/or analytical studies to be conducted well in advance of design. In addition to the collection of meteorological and oceanographical (Met/Ocean) and geotechnical and geophysical data, any of the following types of studies involving physical and/or computer modeling may be required (Shelden and Butler 2003):

- Wave generation and transformations,
- Hydrodynamic currents and tidal flows,
- Sediment transport and pollutant dispersion,
- Vessel maneuvering and navigation, and
- Berthing and mooring loads and berth downtime analysis.

General factors affecting site selection, planning, and layout and design of harbor protection structures, including breakwaters, are given more thorough overall treatment in Agershou et al. (2004), Tsinker (2004), Thoresen (2014), DOD (2001a), and Bruun (1989).

Port facility siting also depends importantly on land support, in terms of storage area and access to landside transportation systems and the many other siting considerations outlined in Table 3-1. Land space requirements range from 2.5 to 5.0 acres per berth for a traditional general cargo terminal to 20 to 50 acres or more per berth for a contemporary container terminal, depending upon the maximum vessel capacity. The proximity of adequately sheltered anchorage areas for vessels waiting for an available berth is another important general consideration.

It is helpful at this point to introduce a few general definitions, but one must remember that many nautical terms may be used loosely with imprecise definitions.

A *harbor* is, in general, a relatively protected water body where vessels can seek refuge from storms, transfer cargo, or undergo repairs. Harbors may be natural, such as well-sheltered bays, estuaries, or river locations, or artificial, such as areas created by breakwaters or dredged basins.

An *impounded basin* is very well protected and can be entered only through locks that maintain a constant water level within the basin.

Coastal basins are formed by encircling breakwaters with narrow but direct connection to the open sea or by dredging into the coastline with access through a narrow inlet.

A port is in general a site that provides facilities for berthing ships for the purposes of cargo and/or passenger transfer and/or for servicing and may consist of a single marine terminal or loading facility occupying a small portion of a harbor, or it may encompass the entire harbor with several terminal or ship service facilities. Offshore ports may exist at exposed coastal locations and typically are dedicated to the transfer of a single commodity type.

An *estuary* is a semienclosed body of coastal water where ocean tidal water mixes with river run-off water. Estuaries may be further classified by geomorphology, such as coastal plain, bar built, or fjords, or by the dominant control of circulation, such as river-, tide- or wind-controlled, or by degree of stratification of the mixing tidal seawater and outflowing river freshwater.

A *channel* is an unobstructed water course of suitable depth for navigation. A restricted channel typically is relatively narrow and bounded by shoreline banks on both sides. An unrestricted channel is a dredged or naturally deep channel bounded by open water of less than the required navigation depth.

A *fairway* is a relatively wide expanse of water with a clear, unobstructed water depth suitable for vessel transit.

A *roadstead* or road is essentially an open anchorage area outside of demarcated channels of suitable depth and width for vessels to swing about at anchor.

A *turning basin* is a widened portion of a channel suitable for turning a vessel around, usually created by dredging.

An *anchorage basin* is typically a well-protected area created by dredging, where usually smaller vessels may anchor.

Within established ports, there usually are defined limits to which construction can take place in order to avoid encroachment on navigable waters. The *channel line* demarks the limits of a dredged or natural channel, which generally is buoyed and of a control depth and width denoted on nautical charts. Inshore of the channel line, many harbors have a designated *pierhead line*, which denotes the limit of open pier construction beyond which no fixed construction is allowed. The pierhead line may or may not coincide with the *bulkhead line*, which is the limit to which solid fill structures can be built.

A *dock* is the most general term for a place or structure at which a vessel may be moored. A vessel is *moored* when it is secured at one location, whether it is alongside a fixed structure or riding to a single anchor. Piers, wharves, quays, bulkheads, and other structures may serve as docks (these terms are further defined in Section 7.1). A *berth* is a location where a vessel can be safely moored, such as alongside a pier or wharf or within a slip between piers or fixed berthing structures, such as dolphins or ferry racks.

Fig. 3-2 illustrates vessel berth exposure conditions, such as within protected harbors, in more exposed semiprotected artificial harbors, and at offshore sea-island-type berths, which are fully exposed and must generally be vacated under adverse weather conditions. Many ports are located within estuaries at the mouths of rivers and also along the river banks well inland. Such facilities may be subject to strong reversing type currents on a regular basis and to more extreme ebb currents and sudden rises in water levels associated with stormwater run-off or dam releases, and some sites may be subject to moving ice floes during periods of spring breakup. Semiprotected and exposed sites may be subject to various types of wave action and water motions, as described in Section 3.5.

Fig. 3-3 illustrates typical berth-type arrangements. If most of the vessel's length must be accessed for cargo handling, the vessel is berthed alongside a pier or wharf structure, which typically extends the full length of the vessel or longer, with mooring hardware distributed along the face of the pier. If the vessel's cargo can be transferred from a single location on the vessel, such as at a tanker's manifold connections, an open breasting dolphin-type berth arrangement may be used, where mooring lines are secured to discrete mooring dolphins connected only by personnel catwalks. Where rapid turnaround and loading operations are required, such as for ferry and roll-on/roll-off (Ro/Ro) operations, a slip-type berth with guide-in dolphins or continuous fender racks may be used. Variations of the three basic berth types include multiple berth arrangements, such as alongside continuous marginal wharves, or between adjacent piers and/or piers set at skew angles to the shoreline. The U.S. Navy (DOD 2005) provides further discussion of the relative advantages and disadvantages of various alongside berth arrangements.

For single-commodity terminals, such as oil terminals and those dedicated to a particular vessel type and limited size range, the layout of the terminal, including fender systems, mooring hardware, loading equipment, manifolds, and cargo deck space, is relatively straightforward. Where a large range of vessel types and sizes is expected, such as at a general cargo terminal, particular attention must be paid to the size and number of vessels and their relative residence times.

Fig. 3-4 illustrates various design considerations in the general layout of a typical vessel berth. Maintaining adequate water depth throughout changing

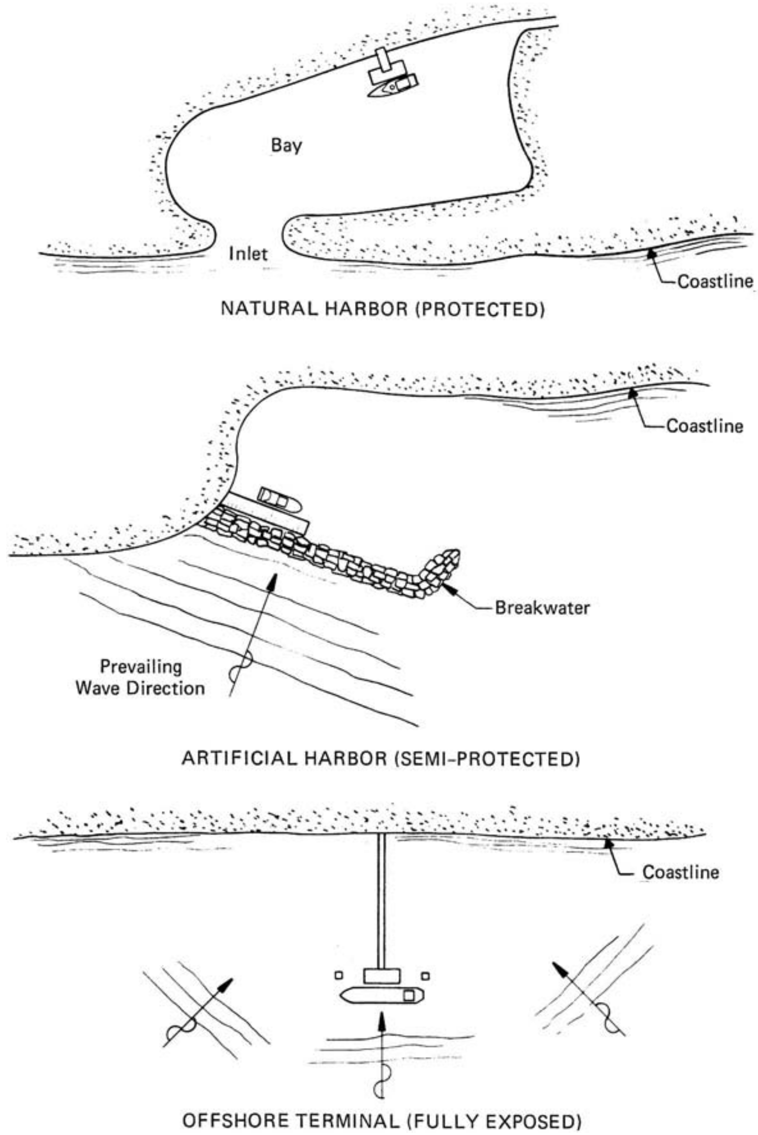


Fig. 3-2. Vessel berth exposure conditions

water levels and vessel draft conditions and consideration of dredge depth and long-term scour and erosion are important to maintaining an adequate under-keel clearance (UKC) for the range of vessels expected. Adequate berth and pier lengths must be provided for the range of sizes and number of vessels to be accommodated. Crane reach and fender standoff distance, as well as adequate pier width and backland area, are important considerations in the early design stage.

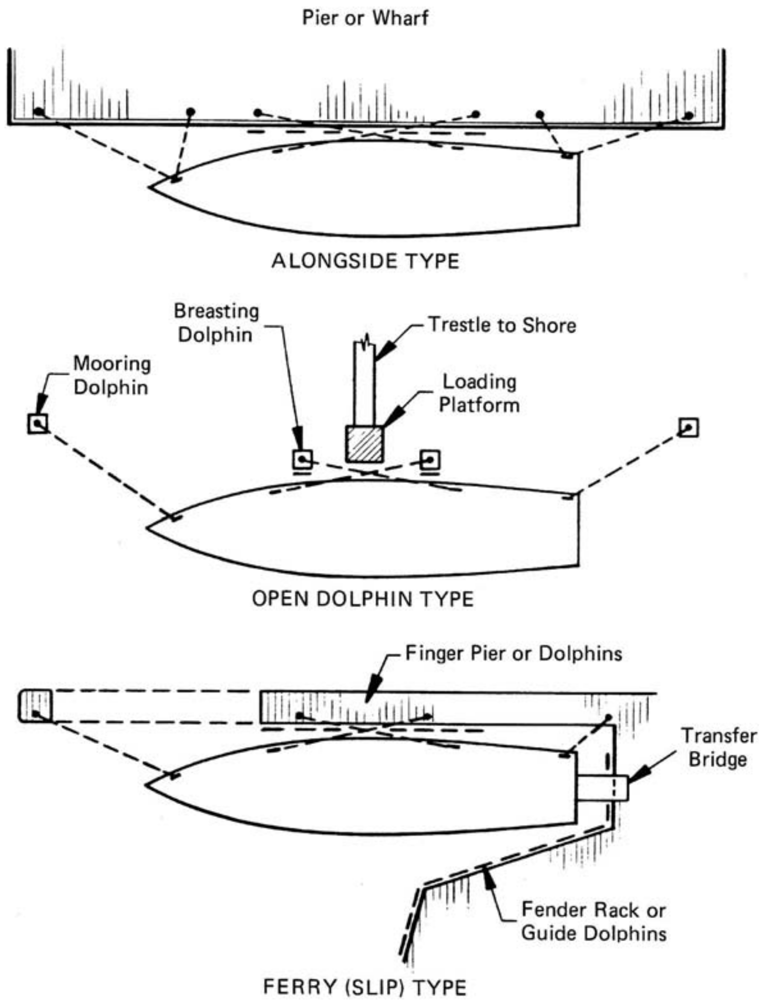


Fig. 3-3. Berth arrangements

Fig. 3-5 illustrates general berth dimensions as recommended by the U.S. Navy (DOD 2005) for multiple berthing. The minimum slip width between adjacent piers should not be less than about 300 ft for larger vessels, or two times the beam of the largest ship plus tug allowance, provided that the vessel can be turned outside of the slip. The slip width also must consider access for smaller service vessels, such as tugs: exposure to wind, wave, and currents; line-handling methods; and the need to move berthed vessels while they are moored. An allowance of from 10% to 20% of the LOA of the largest vessel, or approximately 50 ft minimum, should be provided between vessels berthed alongside, end to end. The term "winding ship in port" refers to the practice of rotating a vessel at a pier so that the opposite side of the vessel is against the pier. Obviously, if this kind of operation is required, then a clear distance equal

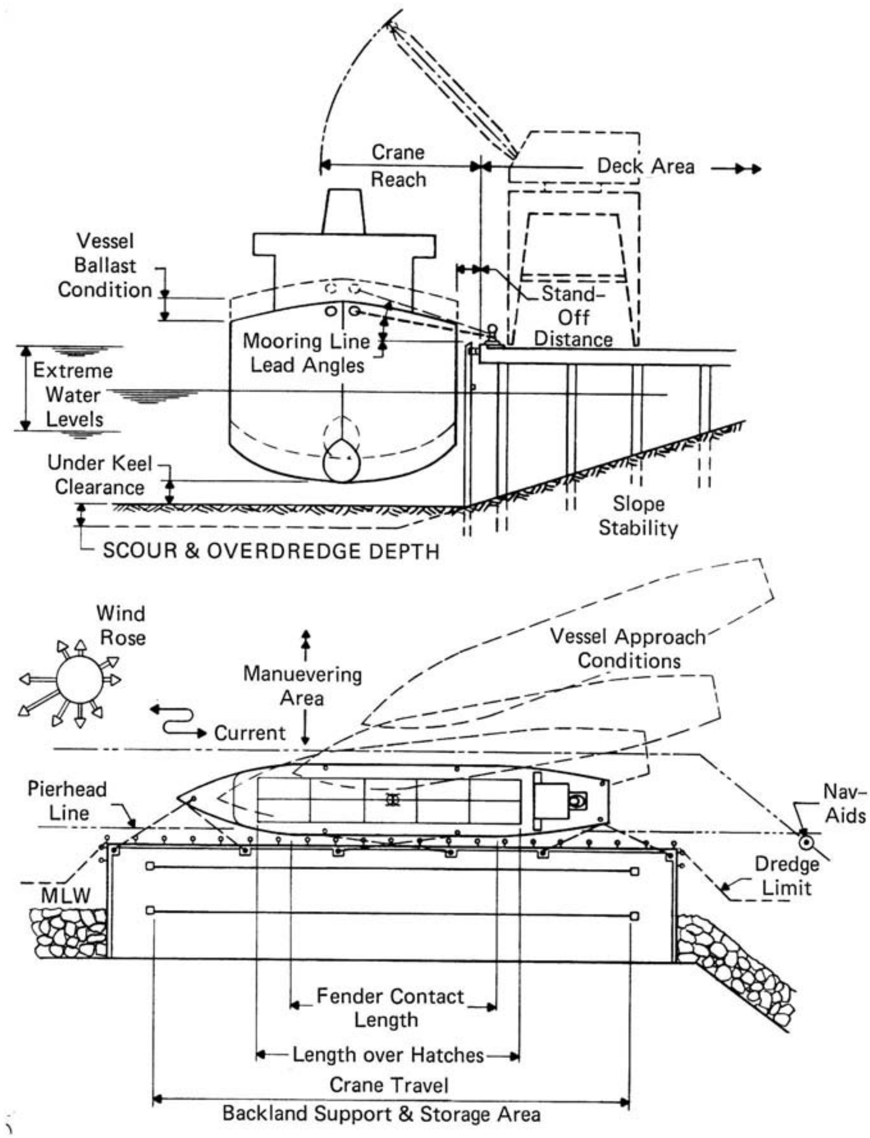


Fig. 3-4. Berth layout design criteria

to the vessel's LOA plus operating room for tugs, perhaps 100 to 200 ft minimum, must be provided. The minimum turning diameter for a vessel being warped around a pier or dolphin is 1.2 times the LOA.

Water depths must have a sufficient under-keel clearance (UKC) with respect to the vessel's full load draft and range of tide. An absolute minimum of 4 ft at low water should be provided to account for vessel trim and tidal variations. Other factors affecting minimum keel clearance include character of the harbor bottom, siltation

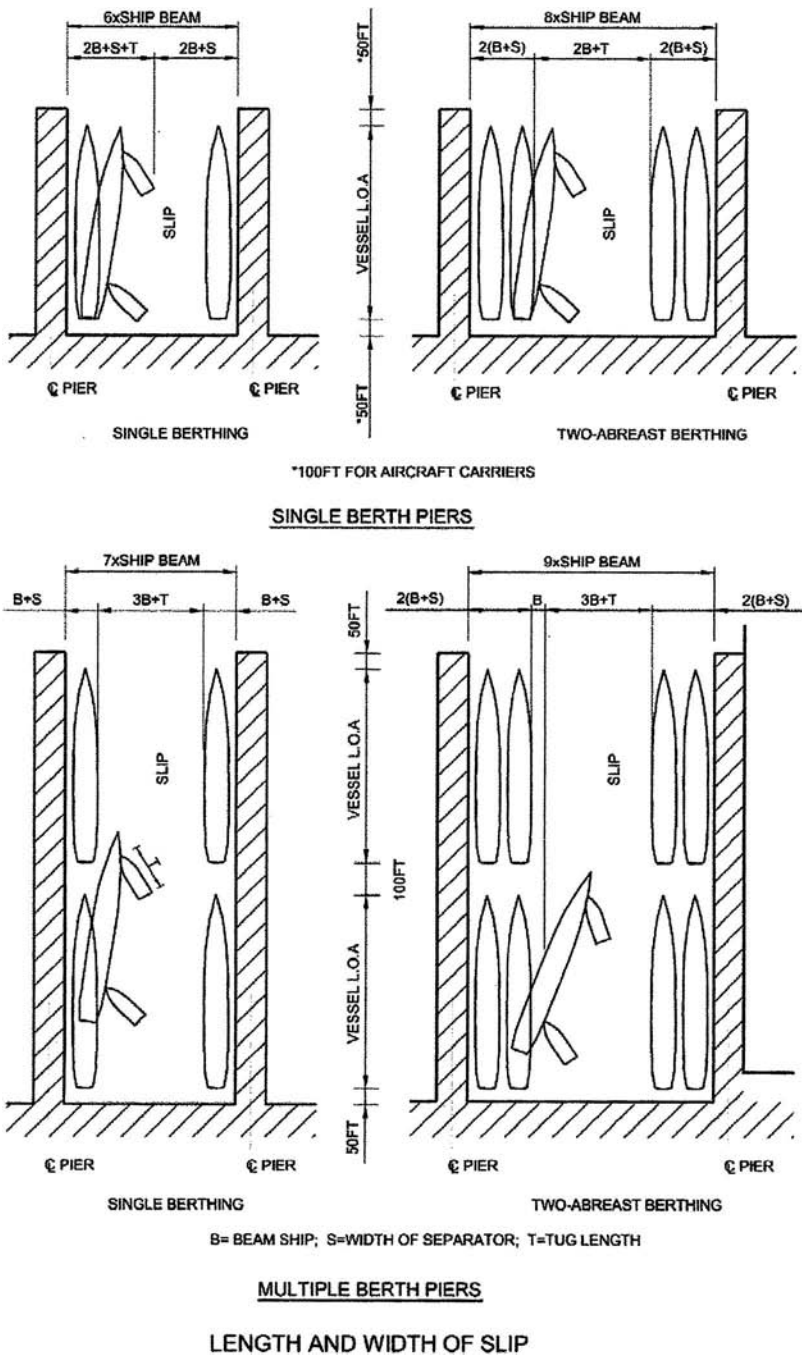


Fig. 3-5. Vessel slip widths for multiple-berth piers

Source: DOD (2005)

allowance, exposure to waves, vessels with vulnerable appendages or water intakes below their keel lines, vessels with extreme trim, and extreme tidal variations. The consequences of grounding on a rock bottom versus soft sediment should be carefully considered in setting the keel clearance, especially in areas with large tidal ranges, which usually have extreme tides that can fall several feet below normal low water. The length of the dredged berth alongside a pier or wharf should be a minimum of 1.25 times the LOA where tug assistance is provided, up to 1.5 times the LOA where tug assistance is not provided. The dredged berth should be at least 1.25 times the beam of the largest vessel to use the berth. For turning basins, a minimum diameter of 4 times the LOA should be provided for unassisted vessels, down to 2 times the LOA where tugs assist.

Dredging and design of navigation channels are important aspects of port and harbor engineering that are outside the scope of this text. Dredging is well covered in the general references of Section 1.5. Design guidance for navigation channels is provided by PIANC/IAPH (2014b); McAnally (2008), McCartney et al. (2005), and Zwamborn (2001) present comprehensive overviews of the subject. Further description of vessel maneuvering area and navigation requirements and other aspects of port engineering can be found in Agerschou et al. (2004), Bruun (1989), Thoresen (2014), and Tsinker (2004). Additional discussion of berth requirements and mooring line layout is provided in Section 6.1 and of pier and wharf general dimensions and features in Chapters 4 and 7. Water depths and channel lines are shown on nautical charts, and navigational conditions and requirements are described in the *U.S. Coast Pilot* on the NOAA and NOS websites (Appendix 3).

3.3 Facility-Type Requirements

Most marine facilities are dedicated to a specific purpose and have particular requirements dictated by vessel type, dimensions, and operating characteristics and by the means of handling and conveying the vessel's cargo. The following factors, which are a consequence of vessel type and cargo transfer operations, determine the facility's overall layout and functional design criteria:

- Berth length and number and location of vessel loading points;
- Apron width and need for short-term storage on the pier or wharf;
- Storage area required, including the need for open backland and covered transit shed areas;
- Cargo-handling equipment requirements and traffic flow;
- Exposure to sea conditions as it affects cargo-handling operations;
- Deck elevation as it affects cargo operations throughout the tidal cycle, and the frequency and consequences of storm flooding; and
- The need for buildings such as storage sheds, waiting areas, customs and administrative offices, and stevedores' shelters and toilets.

In addition to the above considerations, some facility types are particularly adaptable to mixed use, automation, and future expansion possibilities. General trends in the development of modern marine terminals are reviewed in PIANC (1987). Today's ideal contemporary terminal should be adaptable to multipurpose use and able to accommodate the largest loads normally encountered. The following discussion reviews specific functional requirements of the various facility types introduced in Chapter 1. Refer also to Section 2.4 for specific vessel type characteristics.

General Cargo or "break bulk" cargo terminals often receive vessels equipped with their own cargo-handling equipment. The ship's cargo gear is necessarily of limited reach, and cargo is placed on deck near the vessel awaiting conveyance by forklifts, mobile cranes, trailers, or other mobile equipment. General cargo-handling methods are addressed further in Chapter 4 and in DOD (1987, 2005). Long berth lengths are preferred because self-unloading may tie up a single berth for several days. Marginal wharves are preferred to allow continuous access for the mobile equipment and space for operations. Apron widths of 60 to 120 ft are typical. Cargoes may be packaged in a variety of ways, such as on pallets, bailed, bundled, or individually packaged. Even standardized containers may be hauled by break bulk ships. Refrigerated cargoes carried by special reefer ships are often containerized in standard 20- or 40-ft containers with their own refrigeration systems. Contemporary general cargo facilities typically are multipurpose facilities. Minimum single berth lengths of 600 ft or more should be considered. Remote sites, such as isolated offshore islands, are typically required to handle a variety of cargoes with limited facilities (PIANC 2008b).

Container Terminals are rapid turnaround, intermodal facilities that typically require relatively large backland areas for the stacking and temporary storage of containerized cargoes. Typical container dimensions are given in Section 4.1. The containerized cargo system was first used by the Sea-Land Corporation in 1956, and early containerships required a 500- to 600-ft berth and approximately 7 to 10 acres of total yard area. Today, a single berth is generally more than 1,000 ft long and ideally requires 25 acres or more of backland space. Frequently, more than one gantry-type crane loads and unloads the same vessel simultaneously. The apron width is largely determined by the crane gauge plus back-reach and rail setback. Crane gauges vary with their vintage and type but usually fall within the range of 50 to 150 ft, resulting in apron widths of from approximately 80 to 160 ft. In addition to rail-mounted cranes, which load and unload the vessels, various types of rubber-tired mobile equipment, such as straddle carriers, forklifts, and tractor-trailers, are used to move and stack containers (see Section 4.2). Railroad spurs also may be an integral part of container terminals. Optimization of marshaling areas, where containers are stored on trailer chassis, and the traffic flow are critical elements in efficient container terminal operations. PIANC (2014b) provides design guidance for small and medium-sized container terminals, considered as handling less than 250,000 TEU per year and from 250,000 to 750,000 TEU per year, respectively. For comparison, the world's busiest port, Shanghai, handled more than 33 million TEU in 2013, and the top 10 ports handled more than 13 million TEU each in 2013 (Port Technology website data, see Appendix 3, 2014). The Port of Los Angeles (POLA) is the largest port in the United States, and together with its sister Port of Long Beach (POLB) had a combined container throughput of 14.6 million TEU for 2013.

The rapid growth in containership size (see Section 2.4) has driven demand for new higher capacity and upgrading of existing facilities (see Sections 1.3 and 4.3). PIANC (2012b) provides design guidance for container handling facilities in general.

Roll-On/Roll-Off (Ro/Ro) facilities originally were intended as large oceangoing ferries for tractor-trailer service. Vessels usually are equipped with their own ramps, which may be straight stern and/or bow ramps, for end-on loading only, or slewing-type quarter ramps, which allow the vessel to unload from alongside. A variation of Ro/Ro service is the pure vehicle carrier, which primarily delivers new automobiles to their markets. Vehicle carriers usually are equipped with two or more side ramps for self-unloading operation. Today's Ro/Ro vessel often also carries containers on deck, and oceangoing Ro/Ro service requires facilities tailored to this mixed use. Quay and wharf decks must be clear and unobstructed and able to support the localized reactions of the vessel's ramps. The deck elevation relative to the tide range and the vessel's loaded and unloaded threshold height are critical for efficient full-time operation. Quay decks may be sloped to accommodate varying ramp angles and threshold heights through the tide cycle. Many Ro/Ro facilities provide adjustable fixed transfer bridges or floating transfer bridges, referred to as link spans, to overcome the problem of large tidal range. International standards have been developed for dimensioning and details of Ro/Ro ship-to-shore connections (PIANC 1978), and additional information on Ro/Ro ramp and system characteristics can be found in ISO (1983) and ICHCA (1978).

Ferry Terminals service passengers and highway vehicle traffic. They require a small to moderate backland area for short-term queuing of traffic and for passenger waiting areas. Vehicles usually are loaded end-on via transfer bridges similar to those used in Ro/Ro operations. Passengers are boarded through side doors via boarding platforms and gangways, or sometimes over the transfer bridge in smaller scale operations. The most important design considerations for a ferry terminal are its transfer bridge and a high-energy absorption/low-maintenance fender system because of the frequent and high-speed nature of the ferry operation. Ample guide-in dolphins should be properly located to facilitate rapid and safe berthing maneuvers. Another important consideration in ferry terminal design is scour protection from propeller wash, which is an especially acute problem at the toe of bulkheads (see Section 4.6). Additional information on ferry terminal facilities can be found in PIANC (1995) and DOD (2005).

Cruise Ship/Passenger Terminals have experienced a rapid growth in construction over the past decade in concert with the burgeoning cruise ship industry. Alongside berthing space is generally required, and special attention must be given to passenger boarding structures and waiting areas. Because cruise ship piers often have passenger terminal buildings constructed on their decks, they are subject to stricter building code requirements, as for building foundations, as well as fire protection and emergency egress requirements. Separate means of loading vessel stores and provisions also must be provided. Unlike most cargo-carrying vessels, passenger ships do not change draft significantly between loaded and unloaded conditions. A crude rule of thumb requires approximately 10 square ft of space per passenger in waiting areas. Direct connection with ground transportation links is also necessary.

Barge Carrier Systems, generally known as LASH for "lighter-aboard ship" (other, now defunct, proprietary systems include SEABEE and BARCAT), are designed for barge handling afloat where the vessel route typically connects with inland waterways or shallow-water terminals. As such, they generally do not require dedicated shoreside facilities because they are equipped to transfer their cargo while anchored in stream. LASH vessels, however, may berth alongside, especially if they are equipped to carry containers on deck.

Dry Bulk Cargo facilities often are dedicated to handling a specific commodity, such as ores, scrap metals, minerals, coal, cement, grains, fertilizers, or wood chips. They usually are loaded and unloaded by special equipment with conveyor systems to storage and distribution areas. Because of the nature of the cargo and the fact that it can be readily conveyed on a mechanized system, and because of the generally large size and deep draft of bulk carriers, bulk carrier berths are often built offshore. Offshore berths at exposed locations must be designed to survive the most severe sea conditions expected over their design life, as well as maximum vessel berthing and mooring loads under the worst allowable operating conditions with the vessel in berth (see Section 6.1). The majority of offshore-type dry bulk berths are dedicated to loading vessels rather than to offloading because of the greater difficulty of offloading a vessel moving in waves. The storage and distribution area may be a considerable distance from the vessel berth. Very high surcharge or deck loads may result from the piling up of bulk cargoes. Bulk carriers often need to be shifted in position along the berth length to allow ship loading equipment to access all of their holds. Bulk ships undergo a very large change in draft and trim between loaded and unloaded conditions. Traveling-type loading and offloading equipment should normally be provided at high-capacity berths, whereas fixed-location equipment may be provided at lower capacity berths. Some bulk ships, such as certain colliers (coal carriers), are equipped with self-unloading booms and conveyors. Self-unloading vessels may discharge their cargo directly to the quay or wharf deck or into hoppers or bins, or to smaller barges for inland distribution. Some dry bulk cargoes may be mixed with water and pumped through pipelines in slurry form, but this practice is limited because mixing of large volumes of water in a vessel's holds places limitations on its cargo-carrying capacity. To the extent that they somewhat resemble tankers in size and shape, some guidance for the design of large/offshore bulk vessel terminals can be found in OCIMF (2008).

Liquid Bulk Cargo facilities include crude oil and petroleum products (by far the most common) and various chemical solutions and some foodstuffs. Liquefied compressed gases, such as natural gas (LNG), petroleum gases (LPG), and others, are a special type of liquid bulk cargo; they are considered hazardous cargoes requiring low-temperature (LNG) or high-pressure and/or combination of pressurized and reduced temperature (LPG) storage vessels and are subject to strict regulation (ASCE 1976). Design guidance for site selection and design of LNG port facilities can be found in SIGTTO (1997). LNG vessels are similar in berthing requirements to typical crude oil tankers, in that the transfer of cargo usually takes place at a single fixed-manifold location. A typical tanker berth requires only two breasting dolphins flanking a central loading platform for handling hose connections, and additional mooring points ashore or on dolphins for bow and stem lines. It is important to consider the size range of vessels that may use the berth because that range has a major effect on dolphin location and size. Many tankers are of a very large size and thus must berth and transfer their cargo from offshore terminals such as sea-islands or from single-point moorings (SPMs) of various types. The provision of fire protection systems and the need for booster pumps and possible heating of pipelines are important considerations in tanker terminal design. Tankers normally discharge cargo with their own shipboard pumps, but depending upon the distance and the elevation of the storage yard, they may require additional pump stations along the pipeline. Typical tanker pumping rates range from 22,000 barrels per hour (bbl/h) for a tanker of 25,000 DWT to approximately 75,000 bbl/h for a 200,000 DWT tanker. (One barrel equals 42 U.S. gal.) Design guidance for the layout and design of tanker berths can be found in MOTEMS (2011) and OCIMF (2008). For an overview of SPMs, refer to Chapter 9, Sections 9.1 and 9.7. Oil, gases, and various noxious liquids and chemicals are considered dangerous cargoes that are subject to myriad rules and regulations. Important facility design considerations regarding the handling of dangerous cargoes in ports and tanker berth safety are presented in PIANC (2000, 2012b), and design guidance for marine oil terminals in general can be found in MOTEMS (2011).

Ship Building and Ship Repair Yards have unique requirements for new construction and repairs, and shipyards may be specialized in either activity or may carry out both. Ship construction requires large open layout and storage areas, machine shops and administrative buildings, large traveling cranes, and a means of launching, such as building ways, building basins, or transfer onto dry docks. Dry docks are a central feature of ship repair yards and have their own special mooring, access, and operational requirements, as described in Chapter 10. Both shipyard activities require alongside berthing space for fitting out newly launched vessels or conducting repairs afloat. Vessel berths require crane service, usually in the form of revolving portal-type cranes, which typically range from 25 tons to 65 tons capacity. For new construction, revolving-type cranes of 300 or more tons have been built. Fitting-out berths also requires extensive utility, electrical, and ship-services systems, which often have a major effect on the pier or wharf configuration. Fitting-out activities may be carried out from piers with two-side berthing because immediate, continuous access to shore supplies is not constantly required. Access for workers and minimization of deck clutter are important considerations. High-capacity bollards are often required for alongside propeller testing. An overview of shipyard facilities and their layout can be found in BSI (2013b), Mazurkiewicz (1995), and Acker and Bartlett (1980).

Military Bases and Shipyards for the berthing and home-porting of warships are designed to specific naval requirements, such as those given in the DOD unified facilities criteria (UFC), such as DOD (2001a, 2005). Military installations are largely self-sufficient and typically require extensive utility and ship services. In addition, potable water and sewage requirements are high because crews often live aboard vessels while in port, sometimes even during extensive overhauls. Security and fire protection requirements are generally more stringent than for other facility types.

Contemporary warships are typically of lighter hull construction, and fitted camels and/or floating fender systems often are used, in addition to the pier's own fendering.

Fishing Ports and Commercial Service Craft, such as tugs, workboats, small barges, and miscellaneous service craft, usually have minimal berthing facility, backland, and service requirements. Water and electrical power often are the only utilities provided. Fishing vessels require a means of discharging their cargo and loading ice and supplies. This loading and unloading is often done from davit hoists or small jib cranes mounted at fixed locations. Larger fishing vessels may be unloaded by mechanized conveyor-type loading devices. Belt-type conveyors frequently are used to convey fish to the processing facility after they have been offloaded into buckets or containers at the vessel. All commercial small craft generally require relatively protected locations and rugged fender systems, which are subject to continual abuse. Fishing boats and workboats may raft together in congested ports, thus helping to minimize alongside berth length for large fleets. Vessels typically are boarded via vertical ladders built into the side of the wharf or pier structure. Small passenger vessels, such as sightseeing and commuter boats, however, require gangways or direct walk-aboard access, as provided by floating dock systems. Planning and design guidance for fishing ports can be found in PIANC (1998b) and Bruun (1989).

Marinas are small-craft facilities for yachts or recreational vessels that are generally under 50 to 60 ft long. They require sites that are well protected from wave action, as well as adequate backland area for parking and for moving and storing boats. Because the general design of marinas and small-craft harbors is outside of the scope of this text, the reader is referred to the ASCE Manual of Practice No. 50 (ASCE/COPRI 2012), DOD (2009), YHA (2007), CDBW (2005) and the textbook by Tobiasson and Kollmeyer (1991). Refer also to Section 9.7 for floating dock installations. Superyachts, sometimes referred to as "mega-yachts," with lengths generally greater than 80 ft LOA and extreme lengths up to several hundred feet, are ship size and have their own special requirements (PIANC 2013).

3.4 Environmental Conditions

Environmental influences on marine structure design include the effects of wind, waves, and harbor motions; tide and storm surges; currents; ice; water quality factors, such as salinity, dissolved oxygen, pH, and pollutants; climatic conditions, such as temperature range, precipitation, and visibility; as well as fouling and biodeterioration. These factors are discussed in this section, and a more detailed overview of these factors as they relate to structural design has been provided by Gaythwaite (1981). Additional textbook treatment of waves and oceanographical factors and of geological considerations, such as coastal geomorphology, sediment transport, and littoral processes, can be found in the general coastal and ocean engineering literature cited at the end of Chapter 1. Seismic design considerations are treated in Section 4.6.

Environmental loadings are time dependent, covering a spectrum ranging from fractions of a second to several hours. Table 3-3 summarizes the time domains of various environmental influences that are periodic in nature. A dynamic analysis is

Table 3-3. Typical Ranges of Periods (in Seconds) of Marine Environmental Phenomena

Phenomenon	Result	Range of Period
Wind	Turbulence	0.05 to 20[a]
Current	Unsteady velocities in tidal flows	0.1 to 1.0[a]
	Vortex shedding (slender objects)	0.3 to 2.0[a]
	Tidal flows	(same as tides)+
Waves	Wind waves	2 to 20
	Swell	10 to 25
	Long-period waves[b]	30 to 300
	Seiche (harbor oscillations)	20 to 1,000[c]
	Tsunami	600 to 7,000[c]
Tides	Semidiurnal	44,700
	Mixed	to
	Diurnal	89,400
Storm systems/surges		14,000 to 90,000[c] (durations)

[a]Source: BSI (2000).
[b]Infragravity waves caused by storm wave grouping and other causes are typically of low amplitude.
[c]Tidal currents may be rotary or reversing over the tide cycle.

required if the unit response of the structure to any disturbing force is significant, and if any of the structure's natural periods nearly coincide with the period of the disturbing force. Mathematical models for computer application have been developed to describe the various complex interactions of the natural elements, for example, to predict the wave climate within a harbor and for studies of sediment transport and harbor siltation dynamics. General descriptions of mathematical models and computer simulations in harbor and coastal engineering applications can be found in PIANC (2008b), DHL (1985), Jensen and Warren (1986), Fleming (1986), Koufitas (1988), and Yu and Isobe (2001) and applications of physical models in Reis et al. (2014), Hughes (1993), and Davies et al. (2001). Ochi (1993) has presented an overview of wind, waves, and currents as random processes, and their statistical analysis for design of marine structures. The acquisition and application of oceanographic and meteorological data in port design and operations is covered in the PIANC Report No. 117 (PIANC 2012a).

The effects of climate change over the life of a facility should also be considered in the determination of environmental design criteria, with particular regard to sea level rise (SLR) and the possibility of extreme wind speeds and storm surges associated with more intense or frequent storm events. Discussion of climate change issues is beyond the scope of this book. The American Meteorological Society (AMS) publishes an annual state of the climate report that often includes reviews of extreme storm events and current SLR predictions, and current information on climate change can also be found in the literature of the World Meteorological Organization (WMO) and the Intergovernmental Panel on Climate Change (IPCC), see Appendix 3 for website addresses. The following discussion describes the various environmental factors individually and the means of defining them for design purposes.

Wind

Prevailing and extreme wind speeds and directions and their frequency of occurrence at a given site are of utmost importance. Certain sites may be subject to intense tropical storms, such as hurricanes or typhoons relatively infrequently, whereas others may experience less severe but more frequent extratropical cyclones, such as the winter storms common to the midlatitudes. The wind climate at a given site may exhibit regular and dramatic seasonal variations. The local topography may serve to greatly modify wind conditions, resulting in localized funneling or jets. "Gap flow" winds tend to align themselves along narrow channels bounded by steep terrain and may be accelerated by the funneling effect. At some sites in proximity to high hillsides or mountains, local "anabatic" (upslope) or "katabatic" (downslope) winds that may jeopardize moored vessels may be experienced.

In addition to imposing direct velocity forces against fixed structures, equipment, and moored vessels, winds generate waves and may have a profound influence on berthing operations. Of all the factors affecting the berthing maneuvers of large vessels, wind usually is considered the most significant by vessel masters. This is especially true for vessels riding high in ballast condition. Wind data usually are expressed in terms of percent frequency of occurrence and mean speed by direction, usually over a typical 1-year period. This information is conveniently summarized on a diagram called a wind rose. Knowledge of mean annual and seasonal winds is important in berthing and loading operations, especially when there is sufficient fetch for waves to develop, and in other aspects of port operations. For structural design purposes, however, extreme wind speeds associated with longer return periods, such as the 25-, 50-, and 100-year storms, are normally of primary importance. Wind speed normally increases with height above ground within the atmospheric boundary layer, and therefore wind speeds must be reported relative to a vertical datum, usually taken as 10 m or 33 ft above ground. Therefore, it is important to verify the anemometer height of data taken from airports or offshore buoys, for example. In addition, wind exhibits temporal variations, and the averaging time duration of the reported wind speed must also be verified. Wind speeds can then be converted to the appropriate duration for the intended application. In mooring analysis, for example, a 30-s gust is often taken as the basis of design for oceangoing vessels, whereas in wave forecasting (or hindcasting), a mean hourly wind speed or less may be applicable for locally generated wind waves within a harbor; much longer durations of 3 hours and more may be required to properly forecast ocean storm waves.

Fig. 3-6 shows an example of long-term probability distributions of wind speeds over open ocean for several areas of offshore oil-drilling interest. In these curves, the maximum wind is defined as the average of the maximum measured wind speeds over a 1-min interval. The maximum gust is approximately 1.4 times the sustained maximum wind speeds given. The data from which these distributions were plotted were generalized for large areas and are not site specific.

Within harbors and nearer to the coast, extreme wind speeds generally are lower than at offshore locations. Extreme wind speed and direction data for the

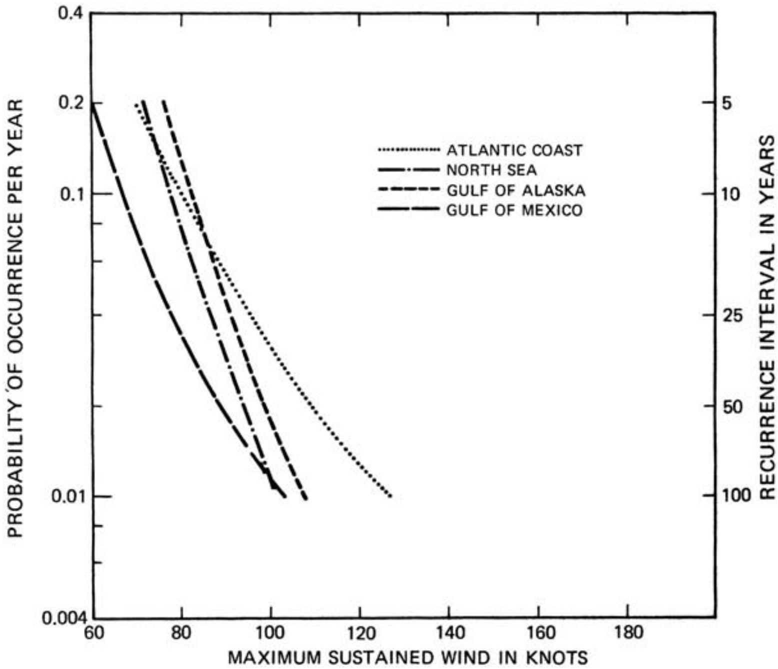


Fig. 3-6. Long-term wind speed versus return period for selected offshore regions

Source: Ochi (1974)

United States and its territories can be found in ASCE (2010), and Batts et al. (1980), DOD (2013), and Seelig and Curfman (2000) give extreme design wind speeds for U.S. Navy installations worldwide. Nearby airport meteorological records maintained by the National Weather Service (NWS) and sometimes from offshore buoys maintained by the National Data Buoy Center (NDBC) are valuable when they can be obtained close to the site. *ASCE 7-10* (ASCE 2010) provides charts showing isolines of basic design wind speeds for the United States and its territories that have been modified to be consistent with building code load factors and specific code criteria but in general are not appropriate for determining mooring loads and wave conditions associated with specific return periods. The Commentary Appendix C on serviceability considerations of *ASCE 7-10* includes charts of actual wind speeds for the 10-, 25-, 50-, and 100-year mean return intervals (MRI) reported as the 3-s gust at 33-ft elevation for open terrain in the United States and its territories. These curves are appropriate for most harbor coastal locations but may need to be corrected for offshore or more exposed open water sites and/or for more inland or developed sites to account for local terrain roughness. Peterka and Shahid (1998) present a means of converting the ASCE wind data to specific MRIs. Refer to the ASCE 7 commentary on wind loads for more in-depth treatment on the development and use of wind data. The Batts et al. (1980) hurricane wind speed data are reported as the “fastest mile” (FM) wind speed at 33-ft elevation for the 10-, 50-, and 100-year return periods. The FM wind speed corresponds to a sustained gust length of 1 mile but is no longer used by the National Weather Service (NWS). These historical data must be properly corrected to appropriate durations for the required use such as in mooring analysis (see Section 6.5) or wave forecasting applications, as described in the following discussions. Recommendations for collecting and interpreting wind data at remote sites where record data are scarce can be found in PIANC (2012a).

Table 3-4 illustrates approximate ranges of threshold wind velocities as they affect marine terminal operations. The wind scale used is the Beaufort Scale, commonly used by mariners to describe wind and sea conditions. The wind speed ranges represent the 10-min average as measured at 33 ft. These operational limits are based on wind action alone. In general, international standards and port and terminal authorities assume that ports remain operational up to around Beaufort force 6 to 8 (van den Bos 2005). In a report on wind influence on container handling equipment, van den Bos (2005) further notes that peak gust wind speeds constitute

Table 3-4. Wind Speed versus Operational Criteria

Beaufort Scale/Seaman's Descriptions		Wind Speed (knots)	Effect on Operations[a]	
			Vessel	Facilities
0	Calm	0–1		
1	Light air	1–3		
2	Light breeze	4–6		
3	Gentle breeze	7–10		
4	Moderate breeze	11–16		
5	Fresh breeze	17–21	↑	
6	Strong breeze	22–27	Berthing limit ↓	↑ Crane operations cease ↓
7	Near gale	28–33	Tugboat limit	↑
8	Fresh gale	34–40		Loading arms disconnected
9	Strong gale	41–47	Ferry operations cease Emergency mooring lines	↓
10	Whole gale	48–55	Larger vessels put to sea	Facilities secured, cranes lashed, etc.
11	Storm	56–63		
12	Hurricane	64–71		

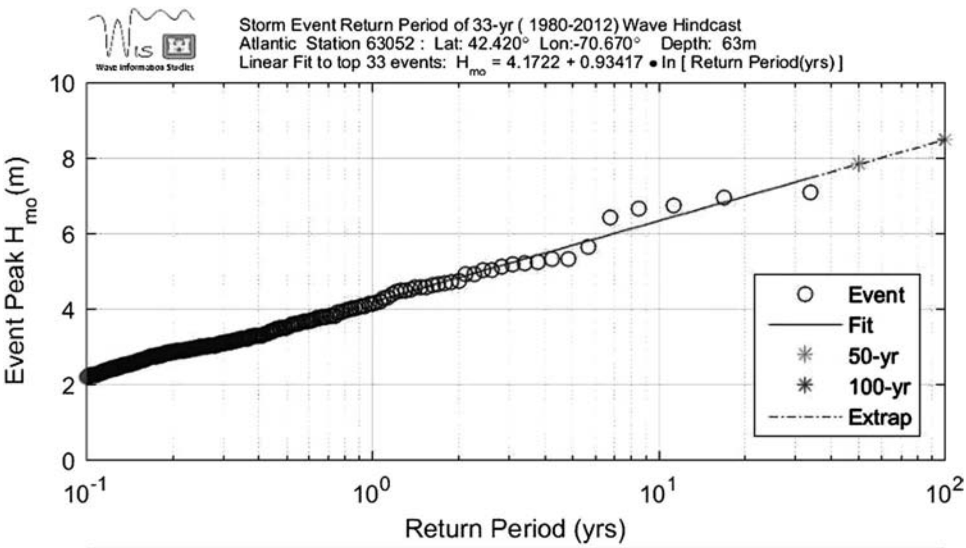
[a]Because of wind alone, wave action at exposed locations may result in greater limitations.

an important threshold level. Container crane operations are affected by sway, and slewing of the containers and side wind forces on empty containers being transported on trailers and by straddle carriers can cause tilting and instability. Vehicle speeds may need to be reduced for wind gusts in excess of around 22 m/s (40 knots), which corresponds to Beaufort force 7 (10-min average). Empty stacked containers that are unlashed may be subject to tilting and sliding at Beaufort force 5. Berthing of large vessels may be affected when winds exceed approximately 20 knots, but high-speed ferries may continue to operate in winds up to approximately 40 knots. Container and gantry-type cranes generally are affected when sustained wind speeds exceed approximately 25 knots. It often is assumed for design purposes that larger vessels will vacate their berths when winds exceed around 50 to 60 knots. This amount should not be taken as a rigid assumption, and the local conditions should be studied carefully to assess the possibility of a vessel's being "trapped" at berth under extreme storm conditions.

Wind speed data and correction factors for design are discussed further in Section 6.4 in connection with the evaluation of mooring loads. The wind climate in general and its effects on structures are treated in detail by Liu (1999), Simiu and Scanlan (1996), and ASCE (1987).

Waves

As with winds, information on both the annual and the extreme wave climate for a given site must be obtained. Most often, wave climate data are compiled from "hindcasting" methods using wind record data and knowledge of the local topography and bathymetry. Short-term data usually are presented in terms of percent frequency of occurrence via histograms and/or cumulative distributions expressing yearly averages for each month. Long-term wave statistics usually are given in terms of some reference maximum wave height versus statistical return period, such as that shown in Fig. 3-7. Where the effects of cumulative damage can be related to the number of wave cycles of a given wave height, as is required in fatigue analysis, then plots of wave height versus cycles over a specified period may be prepared. Such information must include both the normal wave climate and extreme event components. Normally, this information is required only for offshore terminal structures or where the dynamic response to wave action is significant. The wave heights must be applied over an appropriate range of wave periods in order to evaluate the dynamic response in terms of stress level and cycles to failure (API 2014), as discussed in Section 7.3. Another way of defining the wave climate for design purposes is in the form of sea spectra or a wave-height-frequency spectrum. A summary of wave climate data for U.S. coastal waters can be found in the USACE *Coastal Engineering Manual* (CEM) (2006). Wave heights, periods, and directions for 20-year and greater return periods have been hindcast for a dense network of near-coastal and offshore locations under the USACE wave-information study program (WIS). Background on this program and actual measured wave data are given by



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	1991/10/31 05:00	7.09	16.69	75.0	6	2010/03/14 14:00	5.63	11.81	89.0
2	2012/10/29 23:00	6.94	12.78	88.0	7	2003/12/07 03:00	5.32	11.12	62.0
3	1980/01/16 12:00	6.74	12.79	70.0	8	2001/03/06 01:00	5.32	10.01	69.0
4	1992/12/12 22:00	6.65	13.29	77.0	9	1984/03/30 13:00	5.24	11.88	60.0
5	2010/02/26 06:00	6.41	12.70	89.0	10	1981/12/06 16:00	5.19	11.35	46.0

An event is defined as any period when H_{mo} > 2.00m θ_{mean} is direction that waves are arriving from

Fig. 3-7. Example of long-term wave height versus return period from WIS hindcast data for offshore area approximately 15 nm east of Boston

Source: USACE WIS data from website (see Appendix 3)

Hemsley and Brooks (1989). Both real-time and record data for a network of offshore buoys are provided by the NOAA National Data Buoy Center (NDBC). Both NDBC and WIS, as well as other sources of wave data, can be accessed online through the websites listed in Appendix 3.

The three most important parameters controlling wave development at sea are the sustained wind speed; the fetch length, or distance over which the unrestricted wind blows; and the duration of time for which the wind blows. For a given wind speed, the maximum wave height may be limited by the fetch or the duration. If the fetch and the duration are sufficient for a given wind speed, a more or less steady-state condition known as a fully developed sea (FDS) will be attained, whereby average wave heights do not increase any further. It has been well established that within a fully developed sea condition in deep water, the probability distribution of wave heights very nearly follows the extreme-value Rayleigh distribution of statistics (Fig. 3-8). The significant wave height (H_s) is defined as the average of the highest one-third of the waves present in an FDS. The significant wave height may be determined from wave statistics as four times the standard deviation of the sea

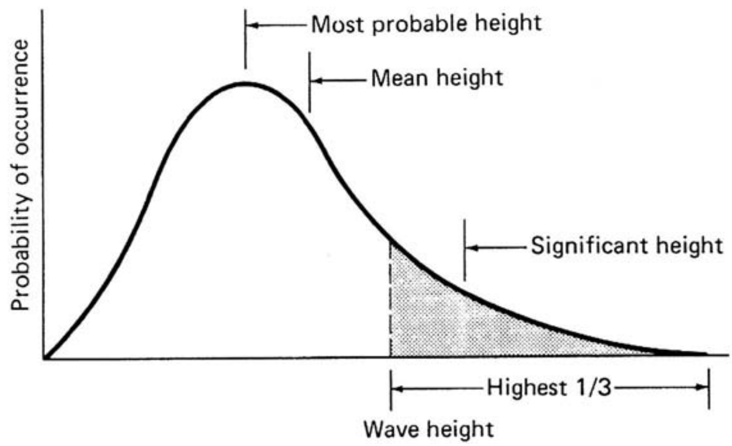


Fig. 3-8. Wave height probability distribution

Table 3-5. Relation of Wave Height Parameters to Significant Wave Height Based upon Rayleigh Distribution

Wave Height Parameter	Relative Wave Height
Average wave height	0.64
Root mean square (RMS) value	0.707
Significant wave height	1.00
Average of highest 10%	1.28
Average of highest 1.0%	1.67
Highest	1.87

surface elevation or four times the square root of the area of the energy spectrum and is usually denoted as H_{m0}. Table 3-5 shows the relationship between H_s and other statistical heights. Note that in reality the highest wave in a given event depends upon the total number of waves in the record (N) as given by

$$H_{\max} = 0.707 H_s \sqrt{\ln N} \tag{3-2}$$

Although this is often taken to be $1.86H_s$, corresponding to the passage of 1,000 waves, it may vary from around 1.6 to 1.8 for a typical storm event to greater than 2.0 in some cases.

Near the coast and in inland waters, wave growth is further constrained by water depth, bottom friction, restricted fetch lengths and widths, and intervening land masses. In the open ocean, fetch lengths and durations required for FDS conditions are seldom attained for sustained wind speeds exceeding 40 to 50 knots, and the developing sea state tends to be steeper and more confused than under FDS conditions. The USACE *Shore Protection Manual* (USACE 1984) provides instructions and useful graphs for estimating wave heights in shallow water that are not included in the CEM. The Automated Coastal Engineering System (ACES) (Leenknecht et al. 1992) is a simple, interactive suite of computer programs, including wind speed adjustment and wave generation based upon the CEM methods. Such calculations can be carried out by an experienced harbor engineer for wind waves within bays or harbors of simple geometry and relatively uniform depths. However, for major facilities and those with greater exposure and complicated bathymetry, wave-climate predictions usually require the use of more elaborate computer programs, such as those provided by the USACE Coastal Modeling System (CMS) (Cialone 1991). These are spectrally based numerical models that solve the wave energy balance equations and include hindcasting models: WISWAVE and WAVAD are used in the WIS studies; the shallow-water, finite-difference, spectrally based program SHALWV is adapted for shallow water and is used in many near-coastal and harbor applications. The Coastal Engineering Design and Analysis System (CEDAS) (Cialone 1991), developed under a cooperative research and development agreement, is a commercially available suite of programs that includes ACES and the CMS and is under continuing development. There are other, often proprietary, programs, such as the Danish Hydraulic Institute's (DHI) MIKE-21 and MIKE-3 program suites, available typically through research universities, and the reader is referred to the literature or to consult a specialist in this field as warranted.

Fig. 3-9 illustrates the basic nomenclature for a simple sinusoidal waveform. The height (H) is the vertical distance from trough to crest, and the length (L) is the horizontal distance between crests. The period (T) is the length of time elapsed between the passage of successive crests. The wave crest elevation above the still water level (SWL) is denoted by η_c. There are various wave theories used to describe wave kinematic properties, and selection among theories depends upon the relative water depth, the degree of accuracy required, and the application. The most basic wave theory is the small-amplitude sinusoidal wave, alternatively known as linear or

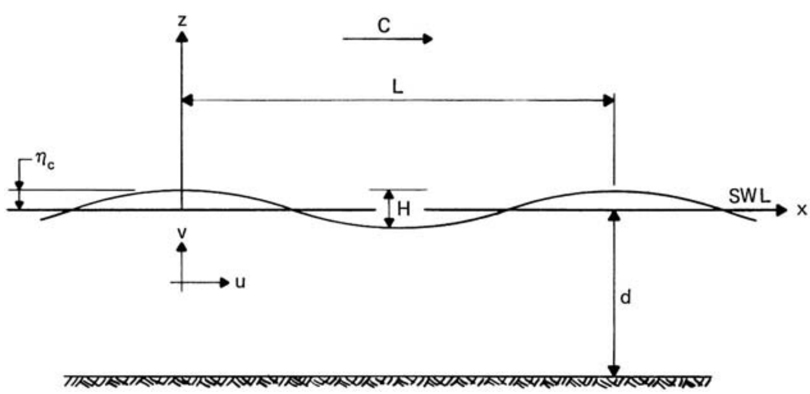


Fig. 3-9. Wave definition sketch

Airy wave theory, named for its originator. According to basic sinusoidal wave theory, the sea surface elevation relative to the SWL at any time (t) is given by

$$\eta(t) = \frac{H}{2}\cos(kx - \omega t) \tag{3-3}$$

where $k = 2\pi/L$ is known as the wave number, and $\omega = 2\pi/T$ is the circular frequency.

The wave phase velocity, or celerity (C), is the speed at which the waveform propagates, given by

$$C = L/T = \sqrt{\frac{g}{k}\tanh kd} \tag{3-4}$$

In deep water, the group velocity (C_g), which is the speed at which a train of waves advances, corresponding to the rate of transmission of energy, is equal to one-half the celerity. In deep water, defined as a depth (d) greater than $L/2$, Eq. (3-4) reduces to

$$C = 5.12T \tag{3-5}$$

for C in feet per second (fps) and T in seconds. Hence,

$$L = 5.12T^2 \tag{3-6}$$

for L in feet. In shallow water, where d is less than approximately $L/20$, the celerity becomes independent of the wavelength and depends only on the water depth, and is found from

$$C = \sqrt{gd} \tag{3-7}$$

where g=acceleration of gravity. In transitional water depths, $L/2 > d > L/20$, Eq. (3-4) applies and, in general, all of the wave properties undergo characteristic changes within the given relative water-depth regimes. As the waveform passes, a hypothetical water particle follows an essentially circular orbit in deep water, which becomes more elliptical in shallower water. The maximum vertical and horizontal particle velocities (v and u) at the surface in deep water are given by

$$u_{\max} = v_{\max} = \pi H/T \tag{3-8}$$

where $u_{\max}$ occurs at the peak wave crest position, positive forward, and the trough position, negative backward (see Fig. 3-8), and $v_{\max}$ occurs at approximately the quarter-wavelength positions upward, positive, with an approaching crest, and downward, negative, after the crest passage. Similarly, the maximum particle accelerations are given by

$$du/dt = dv/dt = 2H(\pi/T)^2 \tag{3-9}$$

v_{max} occurs at approximately the quarter-wavelength position ahead of the crest, positive, and u_{max} occurs at the trough position, positive. Eqs. (3-8) and (3-9) can be corrected for shallow water effects by dividing by kd. The maximum sea surface slope in deep water is given by $\pi H/L$.

Linear wave theory is valid for waves of low steepness in relatively deep water, such as ocean swell. In shallow water and for a more accurate description of real ocean waves, higher order theories have been developed such as Stokes and Stream function theories, for which the reader is referred to the general coastal engineering literature cited. A more rigorous treatment of wave theories and mechanics is beyond the scope of this text. The above relations have been introduced here because of the basic importance of linear wave theory to an understanding of wave phenomena and to subsequent discussions in this book.

Wave periods typically lie in the range of 2 to 20 s, but waves with periods greater than approximately 15 s usually are associated only with severe storms or with swell resulting from distant storms. At sea, wave steepness (H/L) usually ranges from 1/12 to 1/33, whereas near the coast and in inland waters, waves are generally steeper, around 1/9 to 1/17. When waves approach a limiting steepness of approximately 1/17, they begin to break, spilling their energy in turbulence. When waves enter progressively more shallow water approaching a limiting depth, their height begins to increase, and water particle orbits become more elliptical than the circular orbits of deepwater waves before breaking at around the limiting depth of $d_b = 1.28H$. The relative depth at breaking d_b/H is decreased by steep bottom slopes, depending upon the wave's initial deepwater characteristics H_0 and L_0. Wave crest elevations (η_c) above the still water level (SWL) are generally between 55% and 75% of their total height (H) but can approach or exceed 90% under certain shallow-water "solitary wave" conditions.

The waves so far discussed have been assumed to be of uniform dimensions and periods (i.e., "regular" or "monochromatic" waves); however, with the possible exception of some long-period ocean swells, the situation is far different from this idealization for wind waves. A common way of representing the irregular nature of the sea surface is to use a wave spectrum, representing the superposition of a large number of regular waves of varying heights and periods. Wave spectra normally are presented as plots of the spectral density, $S(\omega)$, versus frequency. The spectral density may be given in terms of the square of wave heights or amplitudes, or of the double height or half amplitudes, depending upon the user's preference. The spectral density is proportional to the sea-surface energy. The abscissa of the plot is the wave frequency ($f = 1/T$), commonly given in terms of circular frequency, ω. The area under the spectrum corresponds to some statistical wave-height parameters, such as H_s, when the double-height spectrum is used. For the wave-height spectrum, the area under the curve equals the root mean square (RMS) value of the wave heights present. Several spectral formats commonly are used, depending upon the application. The Bretschneider height spectrum (Bretschneider 1959), for example, is especially useful in many civil engineering applications and was developed to correspond with the Rayleigh distribution of wave heights. Values of H_s and T_s must be determined from wave-forecasting curves or other techniques, whereas other spectral formats, such as the Pierson–Moskowitz (P-M) (1963) and the JONSWAP spectrum (Hasselmann et al. 1973), have wave forecasting parameters such as wind speed and fetch length built into them. The TMA spectrum (Hughes 1984) is especially useful for shallow-water, nearshore applications. Michel (1968, 1999) has presented a simplified review of sea spectra and their applications. Fig. 3-10 shows a family of Bretschneider spectra for a given site under specified wind conditions. The site is somewhat representative of local wind-wave conditions found inside the larger protected harbors of the United States.

Actual measured spectra may be compiled from record data of selected events or time intervals, using spectral analysis techniques such as the fast Fourier transform or autocorrelation method. The measured spectra can be compared with any of the standard spectral formats to see which gives the best representation of conditions at the site. It is not unusual for measured spectra to exhibit more than one spike, especially for distant ocean swell (of very narrow bandwidth) superposed upon local wind-generated seas. Studies of actual measured spectra at a site taken over sufficient periods of time may reveal high levels of energy at unexpected frequencies that could result in harbor or vessel resonant motions.

The wave spectra thus far discussed are essentially unidirectional, describing only the distribution of heights and periods and include nothing about the direction

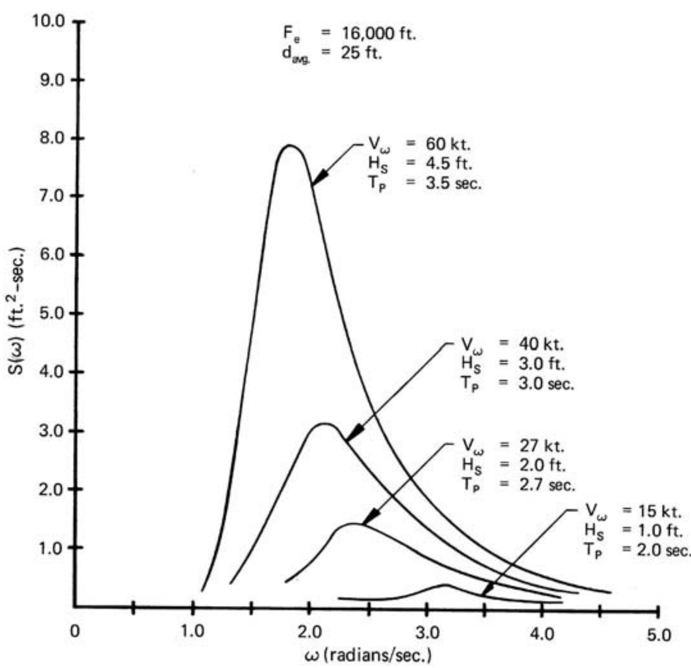


Fig. 3-10. Example wind-wave spectra at protected harbor location

of travel or distribution of height along the crest. Three-dimensional directional spectra (Wiegel 1981) have been developed and applied in certain instances, including hydraulic-model tests, to port structures and harbor design. The directionality of component waves in a given sea state can be accounted for by the application of a directional spreading function. Textbook treatment of random sea processes can be found in Goda (2000) and Sarpkaya and Isaacson (1981).

Long Waves

Wave grouping is another phenomenon that is not described by spectra. There is a distinct tendency for higher waves to occur in groups with some degree of periodicity, which may result in a periodic setup of the water level inside the surf zone, known as surf beats, and a setdown of the water level immediately offshore. The period of wave groups is on the order of five to ten times that of the regular wave period (25 to 300 s or more), which puts it within the usual range of harbor surging and moored-vessel resonance. Such long waves, also known as infragravity (IFG) waves, may be "bound" or "free" IFG waves. Bound waves are nonlinearly coupled to wave groups and travel at the group velocity of the wind waves and are phase locked to sea and swell waves. Free IFG waves radiate to and from deep water after being reflected from the shore and/or when wave breaking interactions are refractively trapped in shallow water and propagate in the alongshore direction. Far infragravity (FIG) waves associated with the setdown under groups of swell waves that have propagated long distances from well offshore are often implicated with problems in ports, such as the initiation of seiche action and resonant response of moored vessels and "setdown," and hence reduced UKC of vessels (Thiebaut et al. 2013). Another form of long-wave resonant phenomena is the "meteorological tsunami" peculiar to certain susceptible sites and associated with atmospheric disturbances, such as gravity and pressure waves and frontal passages that generate *barotropic* ocean waves in the open ocean, waves that are amplified at the coast through specific resonance mechanisms (Monserrat et al. 2006). These waves, sometimes referred to by the Catalan name "rissaga," have similar characteristics to tsunamis but are not as energetic and hence are generally limited to localized effects.

Wave Transformations

As ocean waves propagate into increasingly shallow water, their properties are modified by the effects of refraction, shoaling, and random breaking. *Refraction* refers to the bending and consequent change in direction of wave crests as they begin to "feel bottom" and align themselves with the bottom contours. As waves progress shoreward, the wave motion is retarded by bottom friction in shallow water. Wave heights may be initially reduced but then increased, and their lengths may be decreased by the effects of shoaling. Random breaking of the higher and steeper waves offshore modifies the spectrum of waves before reaching the coastline.

The change in height of a wave caused by refraction and shoaling effects is commonly expressed in terms of the ratio of the modified wave height (H) to the unaltered offshore wave height (H_0), as refraction and shoaling coefficients (K_r) and (K_s), respectively. The shoaling and refraction coefficients are a function of the water depth and deepwater wavelength ratio (d/L_0), and the wave height at any given location must be found by applying K_r and K_s over a series of incremental distances. In general, although wave heights, lengths, and directions may be altered, their periods remain essentially unchanged.

Waves traveling around a fixed object are altered in height and direction by diffraction effects, resulting in waves of diminished height spreading into the area behind the object. Diffraction of large waves around a breakwater can result in notable currents being established along the lee of the breakwater. The diffracted wave height is given by the diffraction coefficient ($K_d = H/H_0$). A useful rule of thumb for waves propagating around the end of a narrow but long breakwater is that $K_d = 0.5$ along an imaginary line drawn from the end of the breakwater in the direction of travel of the incident wave train. Wave heights inside the lee of the breakwater are rapidly diminished, whereas heights actually are increased approximately 10% in a narrow zone just off the tip of the breakwater outside of the imaginary line.

Waves impinging on a fixed object such as the natural shoreline or a structure are partially reflected and partially dissipated, depending upon the nature of the reflecting surface. Reflection coefficients (Krf) range from nearly 1.0 for smooth vertical walls extending above the water surface to around 0.5 for riprap stone slopes of 1 vertical to 2 to 3 horizontal, to approximately 0.1 for natural beaches with relatively flat slopes. When the angle (α) between a line normal to the structure face and the wave crest is greater than 45°, the angle of reflected wave travel equals the angle of wave incidence. When α is between 45° and 20°, there is a Mach-stem-type reflection if the angle between the structure normal and the wave crest is greater than α; and when α is less than 20°, there is no reflection. Under Mach-stem-reflection conditions, wave energy may become trapped and run along the structure, which acts as a wave guide and may result in overtopping. Reflected waves interact with other incoming waves, resulting in waves of greater height, which can be determined from

$$H = \sqrt{H_i^2 + H_j^2 + \cdots H_n^2} \tag{3-10}$$

where H is the resultant height and H_i and H_j through H_n are the heights of the incident waves from various sources. When waves are reflected normal to a vertical wall, they form a standing wave pattern in front of the wall, with a height equal to twice the incident wave height. Waves may pass over submerged barriers or over the top of breakwaters, resulting in transmitted waves of reduced height, as given by a transmitted-wave coefficient (K_t). Fig. 3-11 schematically illustrates the transformation of waves approaching and within a semiprotected artificial harbor.

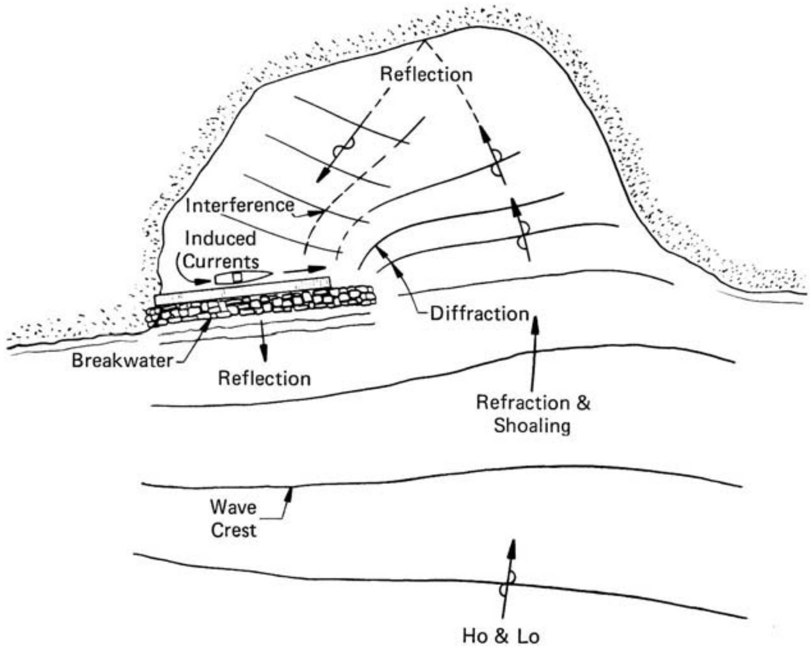


Fig. 3-11. Transformation of wave properties from offshore to harbor location

Wave properties also are modified by currents, being generally steepened by opposing currents and flattened by following currents. As a rule of thumb, waves cannot propagate against a current with a speed in knots greater than three-fourths the wave period in seconds. Modest changes in tidal levels also may result in dramatic changes in wave conditions at a given site. Clearly, the wave conditions possible within a harbor under a range of environmental conditions are very complex.

Harbor Agitation

Wave height criteria must be compared to limiting vessel-motion criteria, which vary with vessel size, type, and orientation to the seas. Limits on vessel motions also may be related to limits imposed by safety considerations, such as risk of damage to vessel and berth and personnel injury, and to limits imposed by operational considerations, such as the ability to efficiently transfer cargo. Allowable vessel motions also may be stipulated by local experience through regulation or local practice. Vessel response is very complex, depending upon the period of the motion as well as its amplitude, as discussed in Section 6.9.

Allowable wave heights in berthing areas may range from 1 ft for small craft to 4 or 5 ft for larger vessels. At offshore moorings where vessels are always oriented head to sea, cargo transfer operations may continue with waves up to 8 to 10 ft high.

Goda (2000) notes that any harbor where H_s is less than approximately 3 ft, even under design storm conditions, can be considered a calm harbor and offers the following recommendations for harbor tranquillity:

1. The harbor should have a broad interior.
2. Any portion of the harbor that can be viewed through the entrance should have a natural beach or wave-absorbing structure.
3. Small craft should not be viewable from the open ocean at any angle.
4. A portion of the waterfront should be wave dissipating in character.
5. Wave reflection from the back side of vertical breakwaters should be avoided, and the use of energy-absorbing quay walls, such as perforated or slit caissons, should be considered.

Wave agitation within harbors is often the result of many interacting sources and modifying boundary effects. In addition to locally wind-generated waves, waves caused by vessel wakes, long-period waves from ocean swell, and harbor resonant oscillations all may contribute to the confused water motions observed in most harbors. Fully enclosed and semienclosed harbor basins, even large bodies of water such as Lake Erie, are subject to resonant oscillations—known variously as seiche, surge, or ranging action—as a result of the water surface being set in motion by some disturbing force, such as wind stress, moving atmospheric pressure systems, incident wave trains at the harbor mouth, and even the movement of large vessels, continuing to slosh back and forth at their natural periods. The sloshing motion may take place about a single nodal point (the point where there is no vertical motion) or may have several nodal points, resulting in a very complicated sloshing pattern. Fig. 3-12 illustrates simplified, typical harbor modes of oscillation.

For an essentially closed rectangular basin of constant depth, the natural period of the first mode (single node) is given by

$$T_n = \frac{2L}{\sqrt{gd}} \tag{3-11}$$

where L is the basin length in the direction of the motion.

For a similar open-ended basin with the nodal point at the opening, the natural period is twice that given by Eq. (3-11). For more complex geometries, refer to Bruun (1989) and USACE (2006) and the general coastal engineering literature. Significant seiches are most usually observed in elongated and narrow inlets and in harbors and bays with narrow entrances. This is the basis of the well-known "harbor paradox" of Miles and Munk (1961), who described why it is that the narrower the entrance and better protected a harbor is from incoming waves and swell, the stronger the seiche oscillations. Most harbors and bays that exhibit such motions have periods in the range of a few minutes to 1 hour or more. Although vertical motion is nil at a nodal point, the horizontal water movement is greatest; and even for low-amplitude seiches, the horizontal displacements of vessels located near nodal

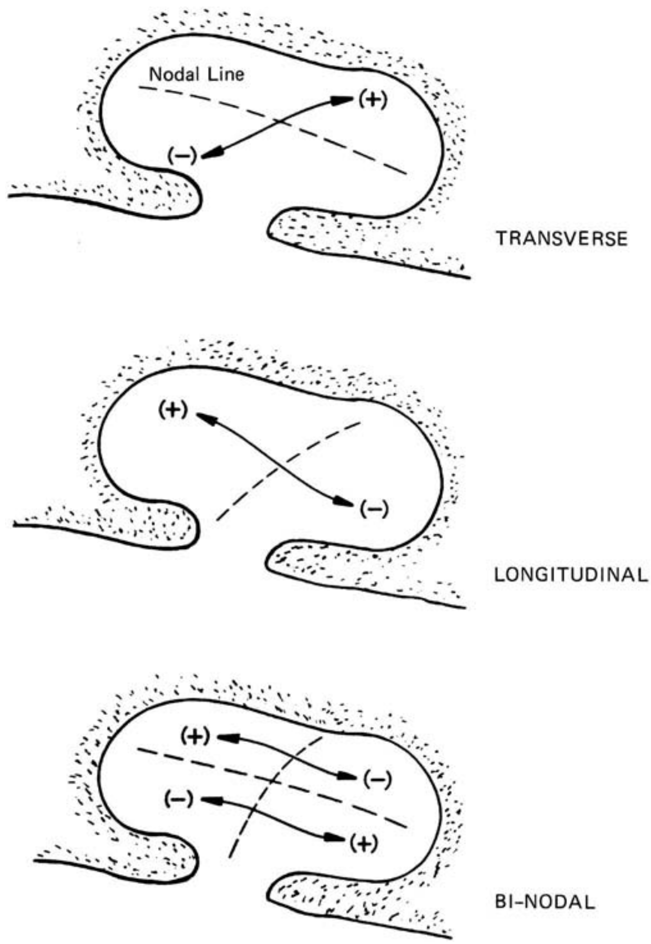


Fig. 3-12. Harbor oscillation modes

points can be very large. Such action has been responsible for the breaking of mooring lines and other damage in larger vessels.

Vessel Wakes

Vessel wakes are a common problem in many crowded harbors, especially for small-craft facilities. Within harbors, wake or "wash" heights are generally less than about 3 ft and of relatively short wavelength, and decrease approximately in inverse proportion to the cube of the distance from the sailing line (Sorenson 1973). For a given vessel size and type, they generally increase with vessel speed and draft for displacement-type vessels and with decreasing water depth. The varying pressure distribution along the length of a moving vessel creates a pattern of diverging and transverse waves that meet at cusp points, as illustrated in Fig. 3-13. The cusp locus

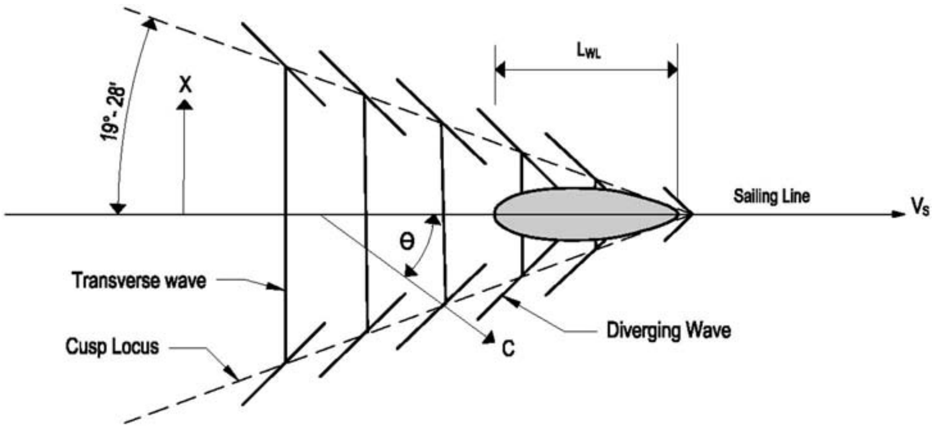


Fig. 3-13. Vessel wake definition sketch

lines meet at an angle of 19° 28′ in deep water, and the angle increases in progressively shallower water and at higher speeds. The diverging waves propagate at an angle (θ) from the sailing line of 35° 16' in deep water. The transverse waves propagate at the vessel speed (V_s), and the diverging wave crests, with an individual celerity of V_s cosθ. Wave heights vary with the nondimensional Froude number (N_F) based on water depth in shallow water and N_F based on vessel LWL in deep water (refer to Section 6.9 for definition of Froude number). Shallow-water conditions can be assumed for depth-based N_F greater than around 0.7. Under such conditions, the wash waves approach the shallow-water wave speed [Eq. (3-7)], and given the propulsion limitations of most large vessels, the vessels' progress is greatly impeded; it is generally assumed that ships cannot operate with $N_F > 0.9$. Vessels operating at high N_F are also subject to sinkage, termed "squat," that is an important consideration in channel design and navigation engineering (PIANC 2014b). Weggel and Sorenson (1986) present a wash-height prediction model, including a simple computer program written in BASIC that is useful in many harbor applications. Guidelines for managing wake wash from high-speed vessels are presented in a report by the Maritime Navigation Commission of PIANC (2003). Vessel wake surface waves described above should not be confused with the passing vessel effects described in Section 6.7, which is a different but somewhat related phenomenon.

Wave Climate Summary

In summary, the wave climate within a harbor is a result of the interactions of waves from the following sources, as transformed by the effects of refraction, shoaling, diffraction, reflection, and the presence of currents.

1. Locally generated wind waves, limited by harbor fetch lengths and water depths;
2. Vessel wakes, limited by vessel size and speed restrictions;

3. Penetration of ocean swell and longer period waves through the harbor entrance, limited by the offshore wave climate and coastal and harbor bathymetry; and
4. Seiches and harbor resonance phenomena, governed by harbor geometry, bathymetry, and meteorological and oceanographical disturbing forces.

Marine facilities at certain locations—the southern Alaska coast and the Hawaiian Islands, for example—may be subject to inundation by seismically generated sea waves, known as tsunamis. These waves are of long length and travel at high speeds at sea. Their heights at sea are virtually undetectable, but when such a wave strikes a coastline, the run-up of water can attain great elevations, and the uprush or velocity forces from the bore that is formed can be devastating; so the potential for tsunami damage at a given site must be addressed (refer to Section 4.7).

Tides and Water Level Variations

Tide is the periodic rise and fall of water levels along the coast in response to the gravitational attraction of the moon and sun, and it may be greatly modified by local topography and bottom contours. The range of tide is an important factor in the planning and design of marine facilities. Tide ranges around the world vary from 1 ft or less at offshore islands to 40 ft or more inside certain constricted bays or basins. Fig. 3-14 shows typical tide curves for several harbors, selected to illustrate a variety of tide ranges and types.

Semidiurnal tides manifest themselves in twice-daily highs and lows of approximately equal height. *A diurnal* tide has one well-defined high and low per day, usually with a much smaller high and low in between; whereas a *mixed* tide falls somewhere in between these extremes. Approximately twice a month, higher-than-average high water combined with lower-than-average low water (i.e., a larger range) occurs, a phenomenon known as "spring tides," caused primarily by the alignment of the sun and moon and consequent enhancement of tide-generating potential. Conversely, twice a month, smaller ranges of tide, called "neap tides," alternate with periods of spring tides. Springs and neaps are usually on the order of 10% to 20% greater or smaller, respectively, in range than the mean-tide range. The sun and moon go through a characteristic pattern of positions relative to the Earth over a period of approximately 18.6 years, known as the "Metonic" cycle. Therefore, tide data must be averaged over such a length of time, known as a tidal epoch, in order to be accurate. The current tidal epoch is 1983 to 2001. Daily tidal predictions for most world ports are readily available on the NOAA/NOS tides and currents website (see Appendix 3). Real-time tidal data corrected for local wind and other disturbing effects plus additional physical oceanographic information are available on the Internet (see Appendix 3) for certain port areas through the NOAA/NOS Physical Oceanographic Real-Time Systems (PORTS).

River tides occur in rivers that empty directly into tidal seas and exhibit notable modifications from ocean tides. Most significantly, the speed at which the tide

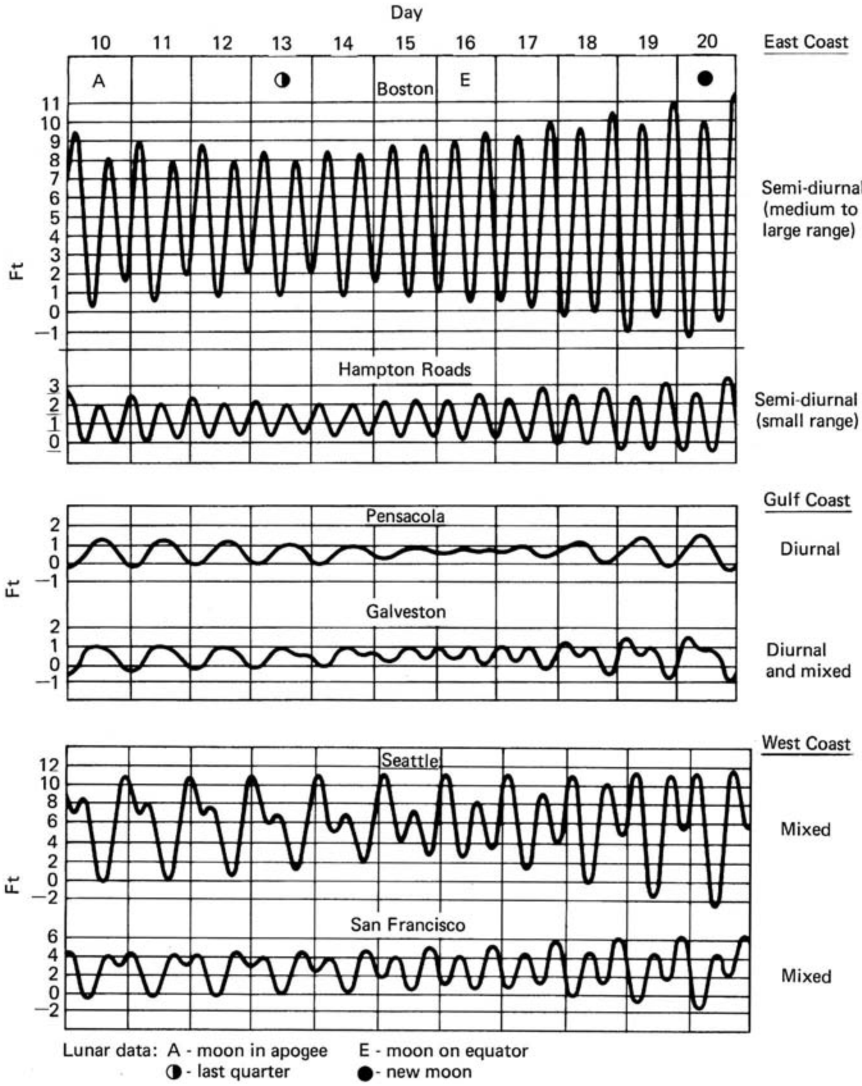


Fig. 3-14. Typical tide curves for selected ports

Source: NOAA, NOS Tide Tables (annual)

propagates upstream depends upon the depth of the river channel; the further upstream, the longer the duration of the falling tide, and the shorter the duration of the rising tide, and the range of tide decreases with distance upstream. River tides may result in strong currents, especially where the river is constricted.

Enclosed inland bodies of water, such as the Great Lakes, exhibit seasonal and long-term variations primarily linked with the timing and magnitude of the major components of the regional water budget. The Great Lakes follow a strong seasonal pattern with relatively low water in the winter months, rising water levels in the spring, and decreasing levels in the late summer and fall. In the 2011 water level year, Lake Erie rose an unprecedented 2.6+ ft between February and June (Gronewold and Stow 2014). Long, narrow, and shallow lakes, such as Lake Erie, may also see substantial short-term water level variations associated with wind stress setup and/or seiche motions. The Lake Michigan–Huron system and Lake Superior constitute the two largest lakes on Earth by surface area; this system rose just over 3 ft from January 2013 to December 2014 after a nearly 15-year period with relatively low historical levels (Gronewold et al. 2015).

Water Level Datums

The establishment of a fixed reference plane is of obvious importance in port engineering. In the United States, nautical charts refer to mean lower low water (MLLW), which is the average of the lowest daily low waters over a tidal epoch. The difference between mean low water (MLW), traditionally used on the U.S. East Coast, and MLLW is typically small for locations with semidiurnal tides, such as the East Coast, but may vary by a few feet at locations with diurnal or mixed tides, such as the West and Gulf coasts. Admiralty charts often refer to the lowest astronomical tide (LAT) for soundings, which is the lowest predicted tide over a long period of time, such as the Metonic cycle. The highest astronomical tide is designated the HAT. In determining design water levels, the average lowest and highest spring tide levels averaged over a long time may be cited, designated the mean low water springs (MLWS) and the mean high water springs (MHWS), respectively. Other tidal reference planes also are in use, and a thorough discussion of tidal datums can be found in Harris (1981), Marmer (1951), and Hicks (1989).

Primarily because of long-term changes in the overall sea level, the mean tide level at a given location varies over time. In the United States, the National Geodetic Vertical Datum (NGVD) was established as a fixed plane of reference in 1929 and corresponded to mean sea level (MSL) at that time. The NGVD of 1929 should not be confused with the North American Datum of 1927, which is cited on nautical charts and refers to horizontal controls. NGVD has generally been phased out by the North American Vertical Datum of 1988 (NAVD-88). Because the NAVD datum is a fixed plane of reference relative to the land, some engineers prefer to reference it on their drawings. The MLLW is typically preferred by most port engineers. It is often helpful to include the current relationship between the MLW and the NGVD/NAVD as well on drawings.

Sea Level Rise

Measurements taken worldwide indicate an overall rise in sea level relative to the land, on the order of about 9 in. per century over the past 100 years, and that the rate of rise is increasing. A National Research Council (NRC 2012) report released estimates of global sea level rise (SLR) of 28 cm $\pm$ 3.2 cm by the 2050s and

82.7 cm ± 10.6 cm by the end of this century. The Intergovernmental Panel on Climate Change (IPCC 2013) notes that historical observations of sea level rise over the past century indicate a 1.7 to 1.8 mm per year rise globally, with an acceleration to 2.8 to 3.6 mm per year over the past few decades. Flick et al. (2003) have reviewed tide data for U.S. coast stations and note that the tide range and, hence, the highest and lowest water levels, has increased at many locations even where the local MSL has remained more nearly constant. It is beyond the scope of this text to speculate on future SLR scenarios, but port and harbor engineers are advised to keep themselves informed of current knowledge of the sea level rise and consider its potential long-term effects in their designs. Guidance for incorporating SLR in civil works design is provided by USACE (2013, 2014), and Ayyub and Kearney (2012) report on an important workshop on SLR and coastal infrastructure. ASCE/COPRI adopted a position paper on SLR in 2014.

Storm Surge

Extreme water levels, denoted as extreme high water (EHW) and extreme low water (ELW), usually are not associated with extreme astronomical tides alone, but rather with a combination of large astronomical tides and storm-surge effects, especially for coastal areas bordered by a wide continental shelf. Storm surges are associated with periods of intense meteorological activity (low atmospheric pressure), which cause a pileup of water against the coast because of an inverted barometer effect, wind stress, deflection of wind-generated currents by the Coriolis force, and, sometimes, resonant effects when the movement of the low-pressure center coincides with the shallow-water wave speed locally. Also, along open coastlines (e.g., beaches), there is an additional setup caused by wave action. The maximum water depth at a site for a given design water level (dwl) then is given by the vector addition of the following components:

$$\mathrm{DWL} = d \vec{+} A \vec{+} W_i \vec{+} P_s \vec{+} W_w \tag{3-12}$$

where

d=water depth at reference level, that is, MLLW;

A=stage of the astronomical tide at the time of peak surge;

W_i=wind setup, consisting of an onshore and alongshore component, which may be deflected shoreward by the Coriolis effect;

P_s=pressure setup associated with reduced atmospheric pressure, which may be greatly enhanced by a resonant response factor at locations where the traveling low-pressure system proceeds along a shallow bay at or near the shallow-water wave speed; see Eq. (3-7); and

W_w=wave setup, which applies only in the surf zone and may be on the order of 10% to 20% of the breaking wave height.

The over arrows indicate a vector addition. The above components usually occur out of phase and are nonlinear. In estuaries and near river mouths, rainfall discharge may also add to the above terms.

A detailed explanation of all these factors is beyond the scope of this text; see Bodine (1971), Hansen (1978), and Bretschneider (1966). Suffice it to say that the increase in water level caused by storm surge alone can be on the order of several feet, as occurs on the U.S. Atlantic (Ho 1976) and Gulf coasts. When this surge coincides with times of high astronomical tide (De Young and Pfafflin 1975), exceptionally high water levels result, allowing larger waves to penetrate farther inland than they normally would. Fig. 3-15 illustrates the propagation of a notable storm surge event along the U.S. East Coast (Parker 2000). In most civilized areas, historical records of extreme high water are available, and they often are presented on a return-period, frequency-of-occurrence basis, as for extreme winds and wave conditions. The Federal Emergency Management Agency (FEMA) has produced Flood Insurance Studies (FIS) and Flood Insurance Rate Maps (FIRMS) for most U.S. coastal communities. The FIS usually gives extreme still water levels (SWL) associated with 10-, 50-, 100-, and sometimes 500-year return periods. The FIRMs give a base flood elevation, which is the maximum wave crest or run-up elevation associated with a 100-year event. Using FEMA methodology (FEMA 1995) for calculating wave heights, the 100-year wave height used by FEMA can be estimated at many locations when

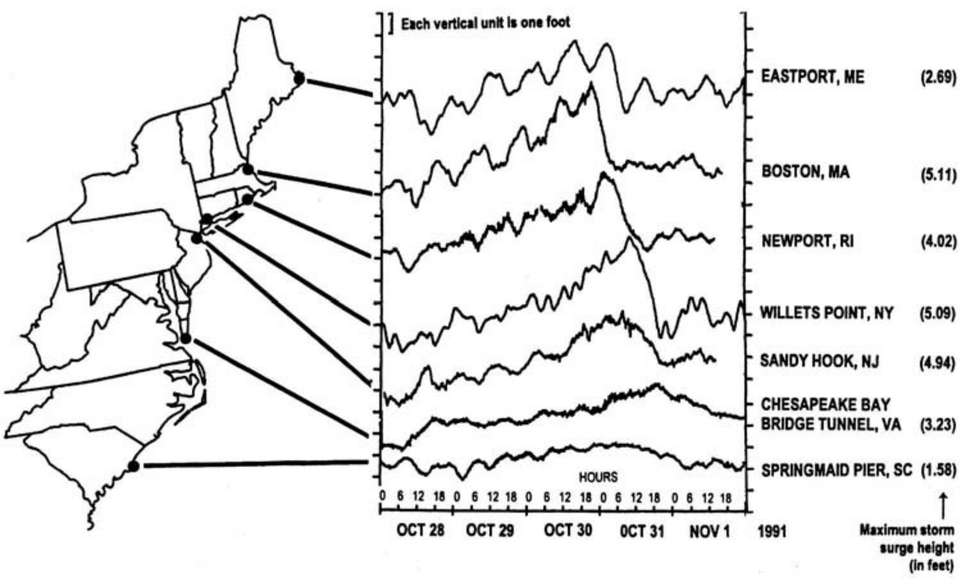


Fig. 3-15. The storm surge (i.e., total water level minus the predicted astronomical tide) during the Halloween storm of 1991, measured at various tide stations along the Atlantic Coast of the United States. The left-hand end of each curve starts at 0 ft (above the astronomical tide), with the exception of the curve for the Chesapeake Bay Bridge Tunnel, which starts a little below zero

Source: Parker (2000)

more detailed wave studies are not warranted. In remote areas, the potential for extreme water levels and associated wave action must be assessed.

Fig. 3-16 illustrates the relationship between various tidal data, storm-surge components, and the design water level for extreme events. In general, extreme water levels may result in overtopping and flooding, increased hydrostatic pressures and buoyancy effects, increased soil pressure and drainage problems, and an increase in the level of action of berthing, mooring, and wave loads. The entire range of water levels must be considered in determining deck elevations and fender system requirements, and in the calculation of soil pressures as well as environmental and mooring loads. Pugh and Woodworth (2014) present a comprehensive introduction to understanding sea level science, including tides, surges, tsunamis, and SLR.

Currents

Current refers to the horizontal movement of water, which is generally associated with the vertical rise and fall of the tide (i.e., tidal currents) or with water level differences, such as river or hydraulic currents. Currents impose velocity (drag) forces on fixed structures and moored vessels, and they affect navigation, transport ice and debris, cause scouring or deposition of bottom material, and affect corrosion rates. Like tides, tidal current velocities vary periodically, being similar to the sinusoidal-like tide curves shown in Fig. 3-14. Currents associated with rising tides are called *flood* currents, and those associated with falling tides are *ebb* currents. Offshore tidal currents are rotary in nature, changing direction through all compass directions throughout the tide cycle. Near the shore and inside bays and estuaries, tidal currents are more typically reversing in nature, flowing in opposite but relatively fixed directions with the ebb and flood. The direction in which a current flows is referred to as the *set*, and its average speed as the *drift*. The time of maximum reversing-type currents is generally near midtide, but currents may lag or precede water levels, so that water movement may be significant even when the tide itself is changing. Current velocities increase (or decrease) in proportion to the increase in tide range such as occurs with spring and neap tide ranges and/or with changes in water levels associated with storm surges or other extreme events. Like tides, daily tidal current predictions are published for many ports (NOAA/NOS tides and currents website; see Appendix 3); however, current velocities usually are reported only in ship channels or at selected reference stations. So in areas where strong currents are noted, site-specific measurements must be taken. Acoustic Doppler current profilers (ADCPs) are commercially available at modest cost and may be mounted on vertical surfaces or set on the seabed to record both speed and direction with variable set sampling rates.

Current velocities usually exhibit a typical boundary-layer-type vertical profile, with the highest velocities at or near the surface, although this is not always the case. Velocities also tend to be maximum along the centerline of a river or channel or follow the "thalweg," which is a line that traces out the greatest depth along the river's length. In some stratified estuaries where freshwater and seawater mix and in

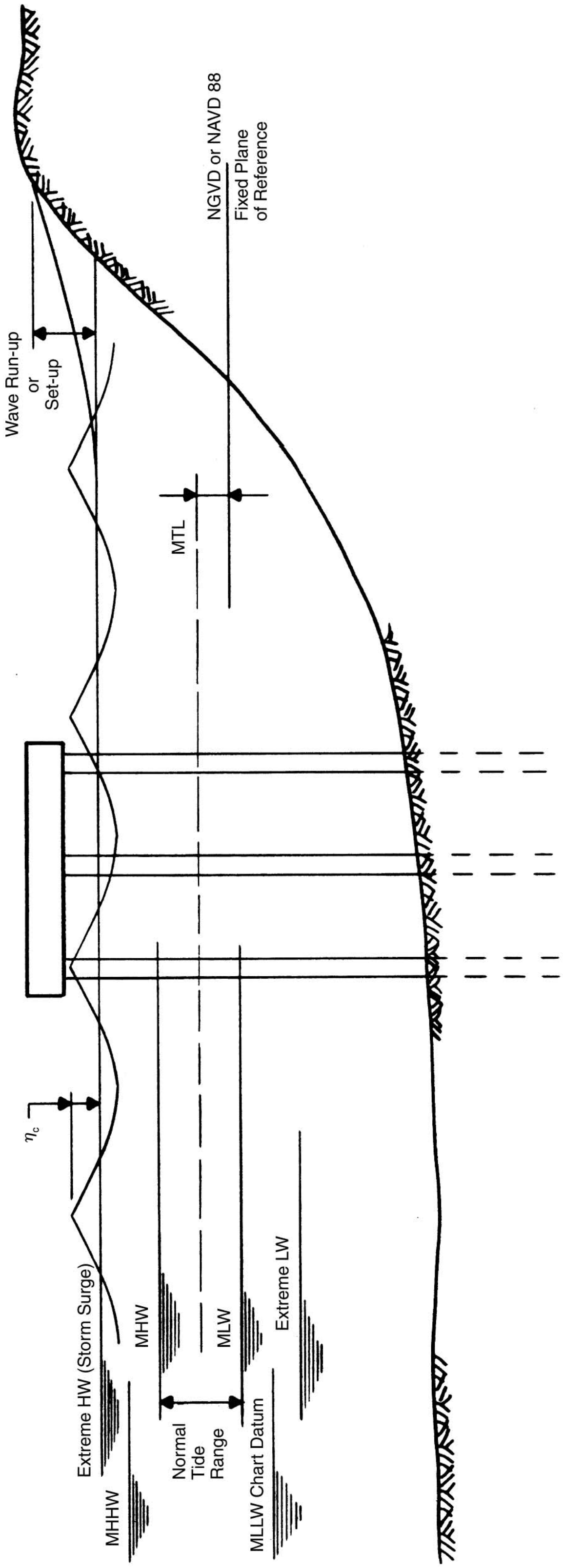


Fig. 3-16. Design water levels

river mouths, a current shear may exist, where the current at the bottom may be in the opposite direction to that at the surface, depending upon the tide stage. Obstructions and changes in flow direction can result in the formation of local turbulence and eddies. The effects of peak river discharges on ebb tides associated with stormwater runoff, dam releases, and spring thaws must also be considered at certain river and estuary sites. Fig. 3-17 illustrates some representative vertical current velocity profiles.

Wind stress currents, usually noticed only under extreme and sustained wind conditions, are primarily surface currents, and for design purposes may be approximated as varying linearly from a maximum of 1% to 3% of the sustained wind speed at the surface to zero at the sea bottom. Wind stress currents may exceed 1 knot under certain circumstances and must be added vectorially to the prevailing tidal currents. Hydraulic currents such as occur in the Cape Cod Canal in Massachusetts result from the difference in water levels at each end of the canal caused by the different times of high and low water and can be very strong because of the restricted waterway width. Turbidity currents result from underwater landslides or slope

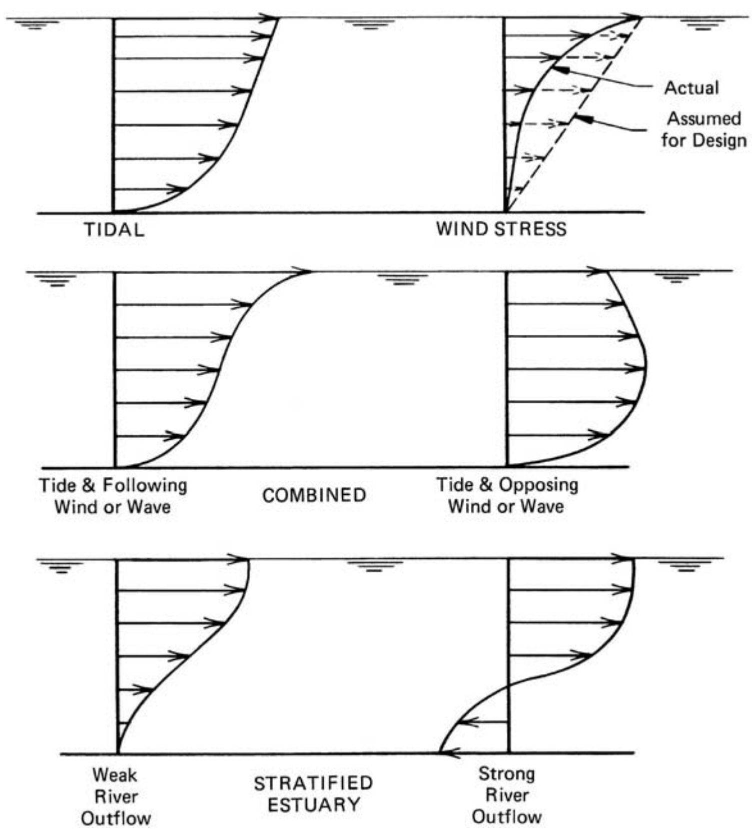


Fig. 3-17. Current velocity profiles

failures and are necessarily restricted in extent and duration but can displace a large volume of sediment suddenly.

It is generally considered that a current of 0.5 ft per second (fps) or more, as measured at 1 ft above a flat bottom, erodes fine to medium sands. The threshold velocity is greater for silts and gravels. Strong currents, especially when combined with wave action, can result in sea floor movements and sediment transport and the formation of *bedforms*, such as sand waves and ripple marks. Seabed scour and scour around piles and structures is of obvious great importance in the design of marine structures and is addressed in Section 4.8. Current loads on structures are discussed in Section 4.5 and on moored vessels in Section 6.6.

Ice

Ice can be a significant factor in the design of facilities located in temperate climates and generally governs the design of arctic structures. The most important parameters determining ice effects on structures are the ice thickness, concentration, persistence, variable mechanical properties, and movement. Ice may exert horizontal thrust, impact, and vertical uplift forces on structures, and it contributes to abrasion and freeze–thaw damage to structures. The U.S. Coast Guard is responsible for monitoring ice conditions in navigable waterways.

Information on ice and its distribution can be found in the numerous publications of the USACE Cold Regions Research and Engineering Laboratory (CRREL) and the International Association for Hydro-Enviroment Engineering and Research (IAHR). The World Meteorological Organization (WMO) also publishes information on sea ice nomenclature and information services. Arctic ice engineering is beyond the scope of this text; interested readers are referred to the proceedings of the International Conferences on Port and Ocean Engineering under Arctic Conditions (POAC) (see Appendixes 2 and 3). Selected references that cover arctic and ice engineering problems in general include Timco (1989), Chen and Leidersdorf (1988), Cammaert and Muggeridge (1988), Eranti and Lee (1986), and Caldwell and Crissman (1983). Wortley (1984) addresses ice problems on the Great Lakes, and PIANC (2004) addresses technical and economic problems with channel icing. Site-specific data on desired ice properties often are difficult to find. Because the mechanical properties of ice are highly variable and ice is an anisotropic material, many assumptions often must be made in estimating ice forces. The design of structures to resist ice forces is discussed in Section 4.5.

Water Quality and Climatic Conditions

Water properties such as temperature, salinity, pH, dissolved oxygen (O_2) content, turbidity, the presence of pollutants, and seasonal variations of all of the above may have a profound influence on the corrosion and biofouling history of a structure. The relationship between temperature, salinity, and density is of fundamental importance in oceanography. Salinity, which is basically the dissolved salt content, customarily is given in parts per thousand (ppt), designated ‰. In the open ocean, salinities range from 32‰ to 38‰; the mean is around 35‰. In coastal waters, the range usually is somewhat less, say 28‰ to 32‰, and in brackish water reduces to that of freshwater. At a mean salinity of 35 ppt and a temperature of 15°C, seawater has a specific gravity (SG) of 1.026. The density is often reported in terms of the density function $\sigma_t = (1 - \mathrm{SG}) \times -1{,}000$. An overview of seawater environmental criteria related to structural design has been provided by Gaythwaite (1981), and instructions for obtaining oceanographic data can be found in U.S. Navy (1968).

Seasonal temperatures, precipitation, and visibility can all have important effects on structure design. Temperature extremes affect ice formation, corrosion rates, and the thermal expansion and contraction of structures. An allowance must be made for runoff and drainage of rainfall. For facilities located along rivers, heavy rains can bring increased water levels and currents. Visibility can be an important factor in berthing vessels and may need to be considered in the selection of design berthing velocities.

Fouling and Biodeterioration

Biological environmental factors that must be considered include the growth and accumulation of certain *sessile* organisms, which affix themselves to a structure, known as *fouling*, and some that directly attack materials such as wood, which they use as a food source or substrate or contribute to the corrosion of metals, known as biodeterioration. Within the immersed and tidal zones, organisms that are generally referred to as marine borers may actively destroy unprotected timber, whereas above the tidal zone, wood may be attacked by fungi and other forms of decay, as discussed in Section 11.2. Various marine bacteria can cause corrosive conditions known as microbially induced corrosion (MIC), which is also addressed in Section 11.2. Although fouling growth generally does not lead to the deterioration of structural materials, it can add considerably to the effective mass and projected areas of structural elements—thus increasing wave and current loads—it may affect the corrosion history of a structure, and it also greatly hampers inspection. Fouling generally can be classified as “soft fouling”—which includes the effects of algae and grasses and soft-bodied organisms, such as tunicates, hydroids, sponges, and anemones—and hard fouling, caused by barnacles, mussels, and other hard-shelled animals. Soft fouling has a specific gravity of around 1.0, whereas “hard fouling,” which can grow to a foot or more in thickness, usually has a specific gravity of 1.3 to 1.4. Fouling growth typically diminishes rapidly with depth below the surface and often follows a temporal sequence that may vary with the season or changes in water properties such as temperature and species abundance. The usual pattern of biosuccession begins with bacterial slimes or biofilms and moves to successively larger and dominant organisms, such as barnacles and mussels, which can significantly increase the surface roughness and projected areas of piles and substructure elements. A more detailed overview of fouling and biodeterioration can be found in Gaythwaite (1981), and general treatment of fouling and its prevention have been presented by Benson et al. (1973), Myers et al. (1969), and WHOI (1956).

3.5 Materials for Marine Construction

Proper selection and application of materials with respect to environmental conditions are of great importance in waterfront and ocean construction. The primary materials used are common to all civil engineering construction: timber, steel, concrete, and stone. Other materials of interest to waterfront engineers include rubber and elastomers used as resilient fendering, bearings, and seals; synthetic materials used for wearing surfaces, pile protection jackets, and geotextiles; composite materials such as fiber-reinforced plastics, used for their corrosion resistance; specialty metals used for special hardware and fastenings, such as irons, steel alloys, stainless steels, and aluminum; and, occasionally, copper–nickel alloys and other exotic metals used for special purposes. Table 3-6 summarizes applications of various materials commonly used in port and harbor construction.

Table 3-6. Usual Applications of Materials in Harbor Construction

Material	Application
Steel	Piles, sheet pile (bulkheads), miscellaneous hardware and fittings, fastenings, grating, pipe, crane rails (use in decking and superstructure generally is limited to deepwater terminal platform structures)
Concrete	Piles, sheet piles (bulkheads), deck systems, walls, caissons
Timber	Piles, sheet piles (bulkheads), decking and deck framing, fendering, mats, cribbing
Stone	Breakwaters, slope protection (use in wall construction generally is limited to existing older structures)
Aluminum	Gangways, ramps, boarding platforms
Cast iron	Mooring hardware and miscellaneous castings
Stainless steel	Fastenings (bolts and pins)
Elastomers and plastics	Fendering, fender facing, bearing pads, wraps and cladding, geotextiles (filter fabrics)
Bituminous materials	Paving, revetments
Masonry and block-work materials	Paving
Specialty items	Chain, wire rope, synthetic fiber lines, protective coatings, grouts

Important considerations in materials selection include the following:

- *Structural properties*, such as density, strength, ductility, fatigue, and impact resistance, and their changes under temperature variations. The material properties also affect the degree of difficulty for field or shop fabrication and installation.
- *Durability*, the natural resistance to the marine environment and other deteriorating agents, as well as protective measures needed and long-term maintenance requirements.
- *Compatibility*, physical and chemical interaction with other materials and the ability to be integrated structurally with other materials.
- *Cost and availability*, including transportation, handling, long-term maintenance, costs, and the availability of required member sizes and desired shapes.

An in-depth treatment of materials for construction of harbor and coastal structures can be found in BSI (2013c), Whiteneck and Hockney (1989), and USACE (1983). The following discussion is intended to highlight some important aspects in the selection and specification of the most common structural materials.

Structural Steels

Specifications for structural steel should consider strength, ductility, fatigue and fracture requirements, weldability and connection methods, and corrosion protection measures. Table 3-7 summarizes various grades of structural steel used in waterfront construction in accordance with the standard specifications of the American Society for Testing Materials (ASTM). As with land-based structures, the most common grade of steel is mild carbon steel, such as ASTM A36, though the yield strength of mild steel has increased to the point where much of the mild steel is now being provided with a 50-ksi yield under a variety of ASTM standards, such as A572 and A992. Although many of the higher strength grades, such as A242, A441, A588, and A690, exhibit varying degrees of improved atmospheric corrosion resistance, their added cost, reduced availability, and lower ductility contribute to their less frequent use, compared to the commonly used steels. ASTM A690 is available in H-pile and sheet-pile sections and is alloyed with 0.52% copper and 0.54% nickel, which give it two to three times greater resistance to corrosion within the splash zone as compared with ordinary structural steels.

Many waterfront structures use steel pipe for structural members and piles because of its low surface area, ability to hold coatings, high radius of gyration, and excellent torsional strength. The pipe is often supplied under ASTM standards A53, A106, A500, or A501. The ASTM A252 standard is intended for pipe piles; however, this standard used alone does not provide much control over the chemical composition of the steel (no limits on carbon content), and this limitation may affect the weldability, grain structure, and mechanical properties of the steel pipe. Pipe may be specified under multiple specifications, including ASTM steel plate specifications for

Table 3-7. Selected Structural Steels for Marine Structures

General Classification	ASTM Description	Description	F_y^a (ksi)	$F_T^{a,b}$ (ksi)	Chemical Composition (%)[c]	Remarks
Ordinary strength (mild steel)	A36	Structural steel	36	58–80	0.20 to 0.29 C max., Al, Co, V, Ni, Mo, Cu for higher strength and toughness	General-purpose, standard structural (carbon) steel, good for formability, weldability
High strength–low alloy (HSLA)	A242	High strength–low alloy structural steel	50	70	0.12 C max., Cr, Ni, Cu, Ti	Atmospheric corrosion resistance 4 to 6 times that of carbon steel without Cu, good coating adherence, good weldability
	A441	HSLA structural manganese–vanadium steel	42–50	63–70	0.22 C max., Cu, V, Ni, Cu when specified	Atmospheric corrosion resistance 2 times that of carbon steel without Cu, good weldability
	A529	High-strength carbon–manganese steel of structure quality	50	70	0.27 C max., 1.35 Mn, P, S, Si, Cu when specified	High-strength nonalloy, generally used in the building industry; previous plate specification, which now includes structural shapes
	A572	HSLA columbium–vanadium steels of structural quality	42–65[a]	65–80[a]	0.21 to 0.26 C max., Co, V, Ni, Cu when specified	High strength, good formability Higher strength grades not recommended for welding. Many steel shapes including H-piles and sheet pile row produced to this standard at grade 50 and 60

(Continued)

Table 3-7. Selected Structural Steels for Marine Structures *(Continued)*

General Classification	ASTM Description	Description	F_y^a (ksi)	$F_T^{a,b}$ (ksi)	Chemical Composition (%)[c]	Remarks
	A588	HSLA structural steel with 50,000 psi min. yield pt. to 4 in. thick	50	70	0.20 C max., Cr, Ni, Cu, V	Similar to A242, atmospheric corrosion resistance
	A633	Normalized HSLA structural steel	42–60	63–100	0.21 C max., Cr, Mo, Ni, Cu, V, Ni, Cb for high strength and toughness	High strength with good impact resistance and toughness, low-temperature service
	A992	HSLA structural steel	50–65	65	0.23 C max., 0.5 to 1.5 Mn, Si, V, P, S, Cu, Ni, Cr	Covers “W” shapes intended for use in building framing
High yield strength (HYS)	A514	High yield strength quenched and tempered alloy steel plate suitable for welding	90–100	110–130	0.12 to 0.20 C max., Cr, Mo, Cu, Ti, B	High strength, superior corrosion resistance. Available in plates only. Heat-treated. Low H_2 welding techniques required
Pipe and tubes	A53 Grade B	Welded and seamless steel pipe	35	60	Similar to A36	—
	A500	Cold-formed, welded, and seamless carbon steel, structural tubing in rounds and shapes	33–46	45–62	Similar to A36	—

	A501	Hot-formed, welded, and seamless structural tubing	36	58	Similar to A36	—
Piles and sheet piles	A252[d]	Welded and seamless pipe piles	30–45	50–66	Similar to A36	—
	A328	Steel sheet piling	38.5	70	Similar to A36	—
	A690	HSLA steel H-piles and sheet piling for use in marine environments	50	70	0.22 C max., Cu, Ni	Superior corrosion resistance within splash zone (A690)
	A709	Structural steel for bridges	36–100	58–130	Various grades equal or exceed A36, A572, A588, A514	Unified specification with good corrosion resistance; can specify impact, toughness, and lowest service temperature

[a] Strength, mechanical properties, and chemistry vary with specified grade and material thickness.
[b] All steels, except HYS, have minimum elongations of >20% in 2 in.
[c] All steels have varying amounts of C, Mn P, S, and Si; other significant alloying elements are listed. Maximum carbon (C) content (%) is also given.
[d] Steel pipe suitable for use as piles is also produced under American Petroleum Institute (API) standards for line pipe.

the basic pipe material from which the pipe is fabricated, or by the use of American Petroleum Institute (API) standards 2B and 5L, with several grades ranging from X42 to X80 steel. When specifying pipe piles, keep in mind that the common sizes of pipe over 24-in. diameter are typically only available in 6-in. increments, and the less common sizes, thicknesses, and grades are more dependent on mill rolling schedules. Pipe pile availability should be verified early in the design process. Consideration should also be made for pile lengths that can be readily transported to the project site, including field splicing and straightness tolerances for long piles. Unless matched ends are specified, the out-of-roundness of some spiral-welded pipe can cause pile splicing problems in the field.

Unlike design of landside structures, waterfront structure designers prefer to stay with more substantial metal thickness as corrosion and overstress allowances for both the members and connecting welds. The use of high-strength steels (100 ksi yield and over) in marine structures is typically avoided because of susceptibility to hydrogen embrittlement, which may be caused by stray currents due to welding, or by active cathodic protection systems (ships or structure). These high-strength steels are commonly used for prestressing or posttensioning strands and for all-thread rod used in tie-rods and rock anchors.

All structural steels and steel hardware used in the marine environment must be protected against corrosion by protective coatings, claddings, galvanizing or metallizing, or cathodic protection measures, as described in Section 11.5. Even where proper corrosion protection is provided, a nominal corrosion allowance, usually on the order of 1/16 in. to 1/8 in. of extra metal thickness, is provided, consistent with anticipated corrosion rates, coating performance, and structure life. In some instances, it is more cost-effective to omit expensive coatings and use even greater steel thickness as a corrosion allowance and to improve pile driving through difficult subsurface conditions.

Miscellaneous Metals

Stainless Steel is most often specified for bolts and hardware but also for miscellaneous structural applications, such as curb nosings and other embedded items. The 300 series are austenitic stainless steels alloyed with chromium (Cr) and nickel (Ni) that have similar strength properties to mild structural steel. These steels have good formability and can be hardened by cold working; when annealed, they are nonmagnetic. AISI type 304 and 316 are best suited to most marine structure applications of interest herein. Type 316 also contains molybdenum (Mo), which increases its corrosion resistance, especially to sulfates, and so is the most often specified. Type 316L, low carbon, is best suited for welded applications. Types 316 and 316L have a yield point of 42 ksi and a modulus of elasticity of 28,000 ksi. Type 304 is known as an 18-8 stainless steel for its 18% Cr–8% Ni content and is a common general-purpose stainless steel with a yield point around 35 ksi. Stainless steel bars and shapes can be specified by type under ASTM A276. Stainless steels offer excellent long-term corrosion resistance in most marine environments but are somewhat susceptible to pitting and crevice corrosion, especially in low-oxygen, reducing environments.

Cast Iron, Ductile Iron, and Cast Steel are used for mooring hardware and fittings and are discussed in Section 6.4.

Aluminum alloys are most often used for gangway and boarding structures and are selected on the basis of a suitable balance of strength, weldability, and corrosion resistance properties. Aluminum is much lighter than steel, with a unit weight of about 173 lb/ft^3 versus 490 lb/ft^3 for steel. Structural aluminum is available as sheets, plates, bars and rods, pipe, and extruded shapes and is usually specified under the 5000 or 6000 series for marine structural applications. Alloy type 6061-T6 is commonly specified for ancillary structures, such as gangways, and has a minimum specified yield strength of 35 ksi, with an ultimate strength between 38 and 42 ksi and a modulus of elasticity of 10,100 ksi. Aluminum has a relatively low modulus of elasticity, about one-third that of steel, and does not have a well-defined yield point. It tears easily under notch stresses, and welding within the heat-affected zone (HAZ) lowers its strength. Aluminum has excellent overall corrosion resistance but must be isolated from dissimilar metals because of its low voltage potential in seawater and is often used as a sacrificial anode (see Sections 11.2 and 11.6). Aluminum can be connected with passive stainless steel bolts or pins, but for mild steel or most other metals, isolating bushings are required. In-depth treatment of aluminum properties and design guidance is provided in the *Aluminum Design Manual* (AA 2010) of the Aluminum Association (AA).

Fasteners and Hardware used in marine construction, such as anchor bolts, chains, and shackles, are described variously in Chapters 5, 6, and 7 with regard to specific applications. Fasteners and hardware are often supplied with a hot-dipped galvanized (HDG) coating (see Section 11.5) or sometimes in stainless steel.

Concrete

Concrete is an excellent material for marine construction when properly made and applied. Concrete specifications should consider strength, permeability requirements, mix proportions, and the use of admixtures as they affect durability requirements, such as resistance to sulfate and freeze–thaw attack. The ACI Committee 357 report, *Guide for Design and Construction of Waterfront and Coastal Concrete Marine Structures* (ACI 2014), is an important guideline for concrete applications in marine structures. Quality control in mixing and placing concrete is essential to achieving the desired properties. Reinforcing steel properties, placement, and details also are critical. Concrete mixes are adapted to special applications, such as cast-in-place structural concrete, precast and precast/prestressed concrete, massive (cyclopean) concrete, tremie concrete, pumped and prepacked concrete, and gunite or shotcrete applications. General design guidance for Portland cements, cementitious materials, and the design and control of concrete mixes can be found in PCA (2011). Table 3-8 lists representative mix

Table 3-8. Typical Marine Concrete Applications and Mixes

Application	Exposure Zone[a]	Minimum Compressive Strength at 28 Days (f'_c) (psi)	Minimum Cement Content[b] ($lb/yard^3$)	Maximum W/C Ratio	Maximum Aggregate Size (in.)	Admixtures/ Remarks
Cast-in-place structural elements	Atmospheric, splash, and tidal	4,000–5,000	611–658	0.40	1–1½	Air entraining
Precast/prestressed structural elements	All zones	5,000–7,000	705	0.40	¾	Air entraining, water reducing
Massive structures	All zones	4,000	611–658	0.40	1½–3	Air entraining, plasticizing pozzolans
Tremie	Submerged	4,000–6,000	705	0.40	3/4	Air entraining, plasticizing
Shotcrete	Splash and tidal	5,000–6,000	705	0.55–0.50	NA	Set accelerator

Note: NA means not applicable.

[a]In general, the more severe the exposure conditions, the greater the impermeability that is required; hence, mix proportions that favor higher strength and density are used.

[b]Cement may be Portland Type I, II, or III, provided that C_3A content is within 4% to 10%. Type II is generally preferred for its sulfate resistance.

requirements for some typical applications. Marine concrete and its applications have been addressed by Marshall (1990) and Gerwick (1985) and in the numerous technical papers collected in ACI Special Publications (1980, 1988). Design guidance for mix proportions, cover requirements, and details can be found in ACI (1997, 2014) and FIP (1985).

The single most important factor contributing to the durability of marine concrete is impermeability. Concrete is most susceptible to freeze–thaw damage, especially in temperate to cold climates with large tide ranges, and to abrasion damage, so a dense, durable, high-quality mix is desired. Structural concrete preferably contains Portland Type II cement for its sulfate-resisting qualities, or Type V cement where high sulfate resistance is required. Types I and III, which often are preferred for precast work, also may be used, provided that the tricalcium aluminate (C_3A) content is within the range of 4% to 10%. Type I (MS) and other mix proportions ensure a durable concrete with adequate protection against corrosion of the reinforcing steel. Admixtures, such as calcium nitrite, may be added to the concrete mix to inhibit reinforcing steel corrosion. Mix proportions must be adjusted to achieve the specific design strength and to meet permeability requirements. The water-to-cement ratio in marine concrete is typically kept below 0.40 to help reduce concrete permeability and slow the penetration of chloride ions and carbonation. In general, the minimum 28-day compressive strength (f'_c) should be at least 4,000 psi for cast-in-place (CIP) structural elements and 5,000 to 6,000 or more psi for prestressed structural elements. Lower strengths may be specified for massive concrete and pile-fill applications, and replacement of a larger percentage of the Portland cement with fly ash can be used to control hydration temperature in massive structures. Higher strengths should be specified where severe scouring action, abrasion, or tide zone exposure exists. Durability is also often improved through the use of supplementary cementitious materials (SCMs), such as fly ash and microsilica, particularly if local aggregates may be prone to alkali–aggregate reactions. The substitution of Portland cement with fly ash SCM pozzolans typically improves durability; however, it results in slower strength gain, so silica fume may also be added to restore early strength gain.

To achieve the above strength and durability requirements for normal structural concrete, the following mix proportions generally apply: the content of cementitious material is within the range of 600 to 700 lb per cubic yard, the maximum water/cement ratio (w/c) is from 0.38 to 0.40, the maximum aggregate size is around 0.75 in., and air-entraining admixtures should be used to obtain air contents of 4% to 8% by volume, depending upon aggregate size and exposure. Air entrainment is essential in freeze–thaw climates. Aggregates should be sound and hard, and other mix proportions should be as required by normal civil engineering practice to obtain the needed strengths and to meet durability requirements (ACI 2001). More massive concrete elements can benefit from larger coarse aggregate sizes, such as 1.5-in. aggregate, which tends to reduce shrinkage cracking and concrete cost.

In addition to strength and durability requirements, concrete mixes may need to be proportioned to meet certain workability and consistency requirements affecting concrete placement, and density requirements for special lightweight or heavyweight applications. Concrete properties can be greatly modified by the use of various admixtures. Superplasticizers are commonly used to improve the workability of low-water/cement-ratio, high-performance concrete mixes; however, when they are used at high dosages to reduce water/cement ratios below 0.40, significant autogenous shrinkage cracking can occur if the concrete is not wet cured (McGovern 2002). Polymer-modified concretes (PMCs) are used in applications where high strength and high bond are required and the mix is to be placed in relatively thin layers. Such applications include toppings, overlays, and various types of patching and repairs. Further discussion of concrete durability, protection, and repair applications is provided in Chapter 11. The reader is referred to the literature cited therein for additional discussion of special-purpose concrete mixes.

Concrete Reinforcing

Corrosion of the reinforcing steel is a common cause of the spalling and degradation of concrete. An adequate concrete cover, typically from 2 to 3 in., over the reinforcement, combined with a dense, durable mix that maintains a high alkalinity at the steel surface, normally suffices for most marine exposures. Where deck surfaces are exposed to road salts and/or to the storage of salts or other corrosive bulk materials, corrosion-inhibiting admixtures or fusion-bonded, epoxy-coated reinforcing steel may be specified, usually under ASTM A775 (green coating) or ASTM A934 (purple coating). There are still questions regarding the bond strength of concrete with the epoxy-coated steel and concerns about accelerated corrosion at locations where the epoxy coating is chipped during handling and installation (Burke 1994), which led to the development of the A934 specification. This specification calls for bars to be bent before coating and has somewhat better abrasion resistance; hence, it is preferred by the U.S. Navy and by the California Department of Transportation for marine applications. Special consideration should be given to bonding when epoxy-coated steel is specified, particularly for substructure reinforcing use. There are some relatively new reinforcing materials that are available and potentially offer improved corrosion resistance, but they have a limited track record in waterfront structures. These reinforcing materials include fiber-reinforced plastic (FRP) rebar (with low modulus of elasticity); carbon-fiber grid, rods, and fabric; stainless steel and stainless-steel-clad rebar; and low-carbide/martensite (100 ksi yield) steel rebar. Design of structural concrete using FRP reinforcing bars is covered by ACI 440.1R-06 (ACI 2006). Synthetic fibers, usually polypropylene, can be used as secondary reinforcement to help prevent shrinkage cracking, to improve impact resistance, and to help resist washout of concrete that is pumped or tremie-placed underwater.

Grout

Grouts have a wide range of applications in marine construction and are usually of the cementitious/hydraulic cement or chemical types, such as epoxy and polymer-modified grouts. Applications can be broadly classed as structural grouts, such as for bedding and leveling of hardware and baseplates and anchorage of postinstalled anchor bolts in concrete, or repair-type grouts, such as for filling of cracks, spalls, and voids and bonding new concrete to existing concrete. Important properties to be considered in specifying grouts include shrinkage and/or length change (ASTM C827), compressive strength, bond strength, and consistency, such as flowability. ASTM C1107 covers nonshrink hydraulic cement grouts for structural load-bearing applications, and the use of grouts in concrete repair applications is discussed further in Chapter 11.

Wood

Timber is the traditional material of waterfront construction. It is relatively inexpensive, durable, and convenient to work with, and it possesses good impact resistance (except at high retention levels of chromated copper arsenate (CCA) treatment) and the ability to distribute loads effectively. Its chief enemies are rot and attack by marine organisms, and, to a lesser degree, abrasion and wear, such as that caused by moving ice or sediment. Southern yellow pine and Douglas fir are the woods most commonly specified for marine construction in the United States. Tropical hardwoods, such as greenheart and ekki (also known as azobe), are desirable because of their natural resistance to borer attack and high strength. Further discussion of timber biodeterioration is presented in Section 11.2.

Timber is specified to meet minimum strength, density, and grading requirements. Moisture content and impact resistance are important considerations. Lumber is graded for quality according to such properties as maximum number of knots, slope of grain, and other characteristics that may affect its strength, in accordance with grading rules established by various agencies depending upon the species. The Southern Pine Inspection Bureau (SPIB) and the West Coast Lumber Inspection Bureau (WCLIB), for example, have grading rules for southern yellow pine and Douglas fir, respectively. Allowable stresses are determined based upon the structural grade, in accordance with the *National Design Specification for Wood Construction* (AFPA 2012). A published standard for allowable design stresses for many timber species, including tropical hardwood, can be found in MOD (2000). The engineering properties of domestic woods are described in greater detail in the *Wood Handbook* by the U.S. Department of Agriculture, Forest Products Laboratory (FPL 2010) and for imported tropical woods (FPL 1970).

Timber treatment is either by oil-type or waterborne preservatives (sometimes both), which usually are applied by pressure methods. Oil-type preservatives typically are either coal-tar–creosote and creosote–oil solutions or preservative chemicals, such as pentachlorophenol, dissolved in a nonaqueous carrier. In the United States today, the use of oil-type preservative treatments in waterfront construction has become quite restricted because of environmental concerns. Waterborne preservatives should be of a leach-resistant type, such as chromated copper arsenate (CCA), of which there are three types depending upon the relative percentages of chromium and arsenic present, ammoniacal copper arsenate (ACA), or ammoniacal copper zinc arsenate (ACZA). The treatment is applied under pressure, usually via the full-cell process, wherein the wood is subjected to a vacuum while the treatment chamber is filled with preservative so as to maximize retention. The most common waterborne preservatives for waterfront construction are CCA and ACZA. In late 2003, the use of CCA preservative for residential construction was phased out and replaced with alkaline copper quaternary (ACQ). This ACQ preservative can be used for above-water marine construction, but it is not recommended for saltwater use. Preservative treatment usually is specified in accordance with the American Wood Protection Association standards: C1-03 for timber products and C3-03 for piles (AWPA 2003). Retentions, which are usually specified in pounds per cubic foot of preservative retained, are described further in the AWPA standards. The specified retention values are measured in the outer layer of the timber, and the interior of piles and large timbers typically used in waterfront construction may have significantly lower preservative retentions. Commonly specified preservative retention values for CCA treatment of southern yellow pine (SYP) and Douglas fir (DF) timber in marine construction range from 0.4 lb/ft^3 above the splash zone and 0.6 lb/ft^3 within the splash zone to 2.5 lb/ft^3 in the tide and submerged zones; for SYP and DF piles, retention values range from 1.5 to 2.0 lb/ft^3 for moderate exposure to marine borers up to 2.5 lb/ft^3 for severe exposure to marine borers. More detailed description of preservatives and recommended retentions can be found in the applicable AWPA standards.

Proper design and installation of fastenings are critical to the construction of sound timber structures. Many of the connections used in marine timber construction can benefit from the use of spike grids and steel and FRP plate and clip angles to increase connection capacity, as discussed in Chapter 11. Also, timber dock structures can be effectively braced using all-thread rebar, rather than the traditional timber bracing, which has low capacity according to modern design standards and is especially prone to marine borer attack and impact damage.

Polymer and Composite Materials

Contemporary alternative materials can be broadly classified as polymer based or composite material (PIANC 2009). Polymers (plastics) may be of the thermosetting or thermoplastic type and are suitable for use in molding, casting, extruding, or laminating processes. Composite materials consist of at least two distinct materials bound together, typically with a polymer resin matrix reinforced with fibers and/or filler materials; glass fibers are among the most common materials. Fiber-reinforced plastic (FRP) composites, or *fiberglass*, is perhaps the most common in marine applications. The raw materials for these alternative materials may be of recycled or virgin materials, resins, fibers, additives and modifiers, and fill and fillers to allow for a wide range of material properties possible. Because of the wide variation in material properties and unfamiliarity with these relatively new materials, they have not been widely adopted but have been making steady progress into marine construction, especially for certain applications. They do however offer important potential advantages, such as resistance to environmental degradation, resiliency and durability, low unit weight with a specific gravity around 1.0, and high versatility in terms of the structural elements that can be produced. Additional design guidance for the use of polymer and composite materials in marine construction can be found in DOD (2001c).

Polymer-based materials can be further categorized into four basic groups relative to their application, as structural shapes, nonmetallic reinforcing, repair and strengthening materials, and hybrid elements or systems (PIANC 2009). Structural shapes include piles, sheet piles, "steel shapes," and dimension "lumber," all of which are most often connected with metal fasteners. The effects of creep, flexural stiffness, and deformations under long-term sustained loading must be carefully evaluated. Cold temperatures may reduce member stiffness. In general, the design of plastic structural members is governed by deflections rather than strength.

Nonmetallic reinforcements are typically fiberglass in the form of chopped fibers or continuous rods or polystyrene rods. Glass-fiber reinforced polymer (GFRP) rebar is the most commonly used rebar in the marine environment, and design guidance for its use can be found in ACI 440.1R-06 (ACI 2006). Deflection and cracking may control where it is used in flexural members because of its lower modulus of elasticity compared to steel reinforcing.

Preservation, strengthening, and repair products include wraps and jackets that are usually filled with concrete grout to preserve piles and FRP sheets or strips that are adhesively bound to the undersides of decks and beams to increase load capacity and prevent shear cracks. ACI 440.2R-08 (ACI 2008) provides design guidance for repair and strengthening of concrete structures with FRP products.

Hybrid structural elements and systems most commonly consist of composite deck systems that have been used for pedestrian bridges and light vehicular traffic.

Plastic lumber has found its way into formerly timber dock construction applications. The large-dimension lumber suitable for dock structures is typically formed from pultruded recycled polyethylene, with or without reinforcing and usually with a foamed interior to reduce raw material quantities and density. The nonreinforced plastic lumber has low rigidity, low bending, and low shear strength (Table 3-9). Some manufacturers have improved the properties of the plastic lumber by adding synthetic reinforcing fibers. Some manufacturers are using FRP rebar to reinforce the plastic lumber and plastic pilings to more closely match the properties of timber. One of these manufacturers also offers plastic pilings with steel rebar reinforcement or internal steel pipe for use as floating "log" camels. There has been

Table 3-9. Properties of Selected Timber and Plastic Lumber Commonly Used in Marine Construction

Timber Type	Moisture Content (%)	Elastic Modulus in Bending (psi)	Allowable Stress in Compression Parallel to Grain (psi)	Allowable Stress in Compression Perpendicular to Grain (psi)	Allowable Stress Shearing Parallel to Grain (psi)	Allowable Stress Bending (psi)	Unit Weight (lb/ft^3)
Douglas fir (southern) [a] No. 1 beams	19	1,200,000	775	350	165	1,300	34±
Southern pine[a] No. 1	19	1,500,000	825	375	165	900	36±
Greenheart (*Ocotea radiaei*)	Wet use	2,500,000	2,060	500	340	3,000	66±
Bongossi/azobe (ekki)[b] (*Lophira alata*)	Air-dry, varies	2,465,000	2,900	1,200	290	3,600	70±
Unreinforced[c] 12 × 12 plastic lumber	Dry	42,000	1,500	—	50–100	250	46±
12 × 12 plastic lumber with (4) 1-in. FRP rebar	Dry	300,000	—	—	—	1,300	49±

Notes: Data are compiled from NDS (2001) design values, British Naval Engineering Std. NES 188, plastic lumber manufacturers, and Tropical Wood Importers data. Timber properties are based on 5-in. × 5-in. or larger timbers in service at more than 19% moisture content.

[a]Pressure treatment may reduce strength properties by as much as 30% to 50% of values given. Refer to DOD (2001c).

[b]Pressure treatment increases unit weight from approximately 2% to 25%, depending upon treatment type and retention.

[c]Based on allowable stress values from one manufacturer using a factor of safety of 2.

a learning curve for the manufacturers of reinforced plastic pilings, including thermal shrinkage cracking and cracking caused by corrosion of embedded steel. The lifecycle cost justification for these materials is based on long service lives, which have not yet had sufficient time to be fully proven in use.

Stone Masonry

Stone masonry, such as granite block, the most durable of all waterfront materials, was widely used around many harbors worldwide until the early part of the twentieth century. Cost and availability greatly restrict its use today in massive structures, but smaller irregular stone still is used as riprap slope protection. When locally available, granite masonry can still be an aesthetically pleasing and cost-effective waterfront construction material for small structures, especially when founded on exposed bedrock, and for repair and rehabilitation of existing stone structures. Large stone blocks can be placed relatively quickly with modern equipment, reducing labor costs, and they can be dowelled together with corrosion-resistant FRP dowels to provide a long service life. Stone is specified to meet minimum density requirements (usually on the order of 160 lb/ft^3), as well as for soundness under chemical attack and abrasion and for water absorption characteristics. ASTM C615, *Specification for Granite Dimension Stone,* can be used to specify granite blocks, and McElroy and Lienhart (1993) provide design guidance for specifying riprap and stone in general. Stone (riprap) slope protection also must meet minimum size, weight, gradation, and placement requirements. Refer also to Sections 11.2 and 11.3 for further discussion of stone masonry construction.

Environmental Degradation

Environmental deterioration usually exhibits both temporal and spatial distribution. The temporal distribution manifests itself in seasonal and long-term variations in marine borer activity, fouling, and corrosion rates, as well as infrequent storm or overload and impact damage, and general wear and fatigue effects with time. The spatial distribution varies with structure type, orientation, and exposure, but generally, on almost all structures a characteristic pattern emerges with elevation relative to water level planes. This general vertical zonation is illustrated in Fig. 3-18, which also shows some typical forms of deterioration found within the defined zones.

The *splash zone* usually extends from the plane of the mean high water (MHW) to the upper limit normally attained by salt spray from spring tides and normal wave action. This zone usually is characterized by heavy corrosion of steel structures and spalling of concrete in freezing climates. Above the splash zone, in the *atmospheric zone*, where salt spray normally does not penetrate, timber structures are subject to rot and general biodeterioration. Immediately below the splash zone, the *tide zone* is defined as the area between MHW and the average or lowest low tide plane, MLW or MLWS. The tide zone is characterized by the heaviest attack on concrete structures,

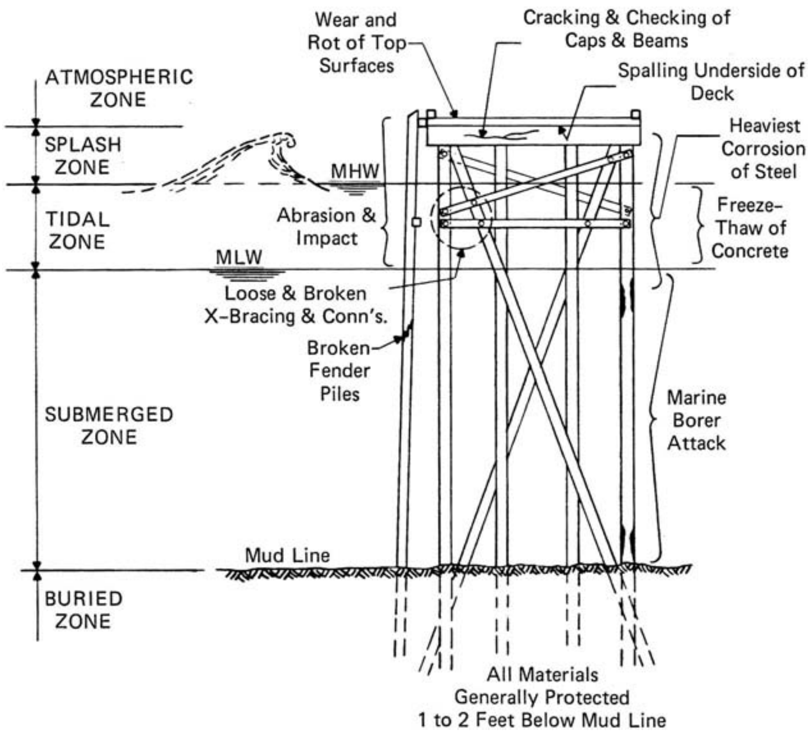


Fig. 3-18. Vertical zonation of deterioration modes

especially freeze–thaw damage in northern climates, and by moderate to heavy corrosion of steel. Below mean low water in the *constantly submerged zone*, corrosion rates usually are moderate, and concrete usually fares well, but timber is subject to attack by marine organisms. Below the mudline in the *buried zone*, most materials are well protected from biodeterioration or physicochemical attack.

In general, the distribution and severity of corrosion and biodeterioration are affected by water properties, such as temperature, salinity, dissolved oxygen content, pH, and pollutants, as well as the tide range and current velocities. More in-depth treatment of modes of deterioration is provided in Section 11.2 and in the PIANC Report of WG-17 (PIANC 2004b). General criteria for waterfront construction with particular regard to material durability is provided by DOD (2001b).

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CHAPTER 4

Operational and Environmental Loads

Piers, wharves, and dry docks typically are designed to support relatively heavy transient loads as compared with other types of civil engineering structures, and a relatively large lateral load capacity is a common feature of waterfront construction. This chapter considers the various sources of direct structural loadings imposed upon marine structures. Because of the importance and complexity of berthing and mooring loads, which often govern the design lateral load capacity of waterfront structures, Chapters 5 and 6, respectively, have been devoted to those loads, and soil pressures and geotechnical considerations are covered in Chapter 8.

This chapter reviews the usual range of deck and cargo live loads, as well as loads caused by vehicles and mobile equipment frequently used on piers and wharves. Loads caused by rail-mounted, fixed, and other specialized material handling equipment also are discussed. Port buildings and various types of elevated superstructures may be constructed on piers and wharves, and environmental forces resulting from wind, waves, currents, ice, and earthquakes may act directly on a structure and its appurtenances. Separate subsections cover these topics. Finally, the effects of tsunamis in ports and other miscellaneous load sources and design criteria that must be considered in the design of marine structures are identified. Load factors and combinations are covered in Section 7.3.

4.1 Cargo Loads

Determination of appropriate uniform live loads and vehicular wheel loads is important to the economic design of waterfront structures. Many existing facilities need to be upgraded in this area because of contemporary transportation practices. In the past, general cargo facilities were designed for uniform live loads on the order of 500 to 600 lb/ft^2 and for standard highway trucks and forklift trucks of perhaps 5 to 10 ton capacity. Today, with containerized cargoes predominant, deck loads of 800 to 1,200 lb/ft^2, truck cranes, and forklift vehicles of up to around 50 tons or more capacity are common. Table 4-1 summarizes representative ranges of deck and equipment loads for various types of facilities. Often, uniform live loads govern foundation design, and wheel loads govern slab thicknesses. Vehicular and

Table 4-1. Typical Deck Loads for Piers and Wharves

Facility Type	Uniform Live Load (psf)	Vehicular Load Class	Rail-mounted Equipment	Special Equipment, Concentrated Loads/Remarks
Fishing and light commercial	250–400	H-10 to HS-20 small F.L.T.	—	Fixed davit/jib boom hoist. Light conveyer.
General cargo	500–1,000	HS-20 10–20 ton F.L.T. 70–140 ton T.C.	AREA E-70 25–50 ton Portal crane	May handle occasional containers, bulk, and palletized cargoes.
Container	800–1,200	HS-20, 30 ton F.L.T. Straddle carrier Transtainer	Container crane	T.C. up to 300 tons may be used at some facilities.
Dry bulk cargo	1,000–3,000	HS-20 Loaders/scrapers	AREA E-80 Specialized loaders/bulk handling cranes	Wide variety of specialized equipment and conveyer systems may be used.
Liquid bulk cargo	60–100 on catwalks and access ramps	—	—	Cargo transfer usually at fixed manifold locations. Design for hose towers, product lines, and other fixed equipment requirements
Ship repair and outfitting	600–1,000	HS-20 70–140 ton T.C.	Portal crane full revolving 25–300 ton	50 ton concentrated load. Extensive utility systems.

Note: F.L.T. = forklift truck: T.C. = truck crane.

equipment loads are covered in subsequent sections. Live-load reduction factors of 25% to 33% of the total uniformly distributed load have traditionally been taken for pile foundation design (Quinn 1972). *Port of Long Beach Wharf Design Criteria* (POLB 2012) requires that wharves be designed for 1,000 lb/ft^2 except that outboard of waterside crane rail, 500 lb/ft^2, and a 20% reduction (800 lb/ft^2) can be taken for the calculation of pile loads. Impact factors do not normally apply to uniform live loads.

Table 4-2 presents typical cargo weights for various materials and stacking heights. General cargo occupies, on average, around 70 ft^3 per long ton (l.t.), corresponding to a mean density of 32 lb/ft^3 and normally ranges from around 20 ft^3/l.t. for tightly packed, high-density items to 140 ft^3/l.t. for loose, low-density items. (Seawater occupies 35 ft^3/l.t.) Uniform live-load criteria and design loads for miscellaneous materials can be found in ASCE (2010), DOD (2005), BSI (2000, 2013), and PIANC (1987a). Quinn (1972) suggests designing the floors of general cargo storage areas for around 500 lb/ft^2 for most purposes, with facilities for specialized goods designed for more or less, such as wool and cotton for 300 to 400 lb/ft^2 and metal products for 600 to 800 lb/ft^2. General cargo can be shipped in boxes, crates, bales, barrels, and so on, or with no packaging at all.

One common way to bundle general cargo is to use pallets. A *pallet* is essentially a wooden or metal base frame upon which the cargo sits, and it usually is secured with metal banding. The pallet facilitates lifting and transport by forklift vehicles and usually has provisions for being lifted by slings. Pallets generally are loaded to a maximum height of around 8 to 10 ft but may be stacked up to approximately 16 ft under certain

Table 4-2. Typical Cargo Weights versus Stacking Heights*

Cargo/Packaging	Packaged Density (lb/ft^3)	Stack Height (ft)	Live Load (lb/ft^2)
General cargo, average/on pallets	32	8–16	256–512
Timber/bundled:			
Softwoods	to 40	10–20	400–800
Hardwoods	to 72	10–20	720–1,440
Paper/baled	50	to 10	500
Dry goods: cotton, wool/baled	20–50	to 10	200–500
Produce: fruits, vegetables, grains in bags, cartons, and cases	31–58	to 10	310–580
Fertilizers in bags	52–59	to 6	312–354
Cement and lime in bags and barrels	50–73	to 6	300–438
Oils and paints in barrels and boxes	35–70	to 6	210–420
Rope (Manila) in coils	42	to 6	252
Metal products in loose bars and coils	75–225	to 10	750–2,250
Automobiles	7.5–15	—	15

Note: Representative weights and stack heights are given. Specific applications should be investigated independently where storage loads are critical. Values given were averaged from various sources.

Table 4-3. Typical Standard Sizes of Pallets (in.)

Rectangular	Square
24 × 32	36 × 36
32 × 40	42 × 42
36 × 40	48 × 48
32 × 48	
36 × 48	
40 × 48	
48 × 60	
48 × 72	
88 × 108	

circumstances. Pallets can be of any size. Some typical standard sizes (in inches) per the U.S. Standards Institute (USSI) are shown in Table 4-3. Palletized cargoes normally require deck load capacities around 400 to 600 lb/ft^2 for stowage of unstacked pallets.

General cargo is most commonly shipped in *intermodal* containers, which usually are handled by special shoreside cranes and ultimately transferred to or from trailer trucks. Containers may be of various sizes, usually 8 ft wide by approximately 8 ft high and from 10 ft up to 48 ft long. They may be constructed of either aluminum or steel, and empty "tare" weights range from approximately 6,200 to 7,900 lb and from 4,200 to 4,800 lb for standard 40-ft and 20-ft units, respectively. Smaller containers are also in use, such as the traditional Conex boxes used by the U.S. military. Various standards organizations, including the U.S. Standards Institute (USSI) and the International Organization for Standardization (ISO), as well as shipping companies, have been involved in the standardization of container dimensions; see Table 4-4 for some common sizes. The most common standard 20-ft equivalent (TEU) shipping container (see Section 2.4) has the following general dimensions, which may vary slightly by manufacturer: 19.9 ft long × 8.0 ft wide × 8.5 ft high with an interior volume of 1,169 ft^3 and a gross weight of 53,000 lb, including an empty tare weight of up to 5,400 lb. Heavy tested 20-ft units are available with maximum gross weight of 66,139 lb, which includes a minimum empty weight of 4,840 lb. The majority of shipboard containers, approximately 90%, are 40 ft long (2 TEU) × 8.0 ft × 8.5 ft with an interior volume of 2,720 ft^3, a gross weight of 67,200 lb, and an empty weight of about 8,800 lb. The TEU is not a precise unit of measure because 20-ft-long containers may be of various widths and heights and are still considered TEUs. The weight capacity of a given size container may vary within regulated limits as well.

Loaded containers can weigh from 5 l.t. to more than 30 l.t., corresponding to approximately 210 lb/ft^2 for a single container and 420, 630, and 840 lb/ft^2 for containers loaded to capacity and stacked two, three, and four high, respectively. Container handling facilities usually are designed for a minimum of 800 to 1,200 lb/ft^2, and the deck system design may be governed by the high wheel loads

Table 4-4. Dimensions and Capacities of Intermodal Containers

Exterior Dimensions (ft)	Approximate Volume Capacity (ft^3)	Approximate Load Capacity (lb)	Maximum Gross (metric tons)
$8 \times 8 \times 20$	1,040	40,000	24
$8 \times 9 \times 20$	1,150	40,000	24
$8 \times 8 \times 40$	2,090	45,000	30[a]
$8 \times 9 \times 40$	2,290	45,000	30[a]
$8 \times 8 \times 45$	2,300	45,000	30[a]
$8 \times 9 \times 45$	2,500	45,000	30[a]
$8 \times 8 \times 48$	2,400	45,000	30[a]
$8 \times 9 \times 48$	2,600	45,000	30[a]
$8 \times 9 \times 53$	Not transported by vessels		

[a]Port planners and designers should assume 35 metric ton gross weight for these units.

of container-handling straddle carriers and forklift trucks (to be discussed). Even general cargo vessels frequently carry some containers on deck, so such facilities should be designed with limited container handling and stowage capacity in mind. Accordingly, most contemporary general cargo facilities should be designed for a minimum uniform live load of 600 to 1,000 lb/ft^2 and multipurpose container roll-on/roll-off (Ro/Ro) facilities for a minimum of 1,000 lb/ft^2 (PIANC 1987a).

For fishing and light commercial use, including passenger and ferry use, a uniform live-load capacity of from 250 lb/ft^2 to perhaps 400 lb/ft^2 usually is sufficient. Piers limited to pedestrian use, such as marina access, commuter boat piers, and so forth, should be designed for a minimum of 100 lb/ft^2. Where automobiles or light trucks are allowed, a uniform live load of 200 and 250 lb/ft^2 should apply. Gangways up to around 4 ft wide should be designed to safely accommodate a minimum of 40 lb/ft^2, and wider gangways, 60 to 100 lb/ft^2. In general, catwalks and access ramps should be designed for from 60 lb/ft^2 to 100 lb/ft^2, depending upon the application, the designer's judgment, and local building code requirements. AASHTO (2009a) requires 85 lb/ft^2 over walkway surfaces of pedestrian bridges. The U.S. Navy (DOD 2005) requires that brows (gangways) and boarding platforms be designed for a minimum of 75 lb/ft^2 and further limits the deflection of brows to span $\times$ 1/240 under full live load plus dead load conditions. Such structures should also be checked for local concentrated loads, depending upon their intended use. See Sections 7.7 and 9.6 for further discussion of gangways and walkway structures.

Concentrated loads, other than wheel loads, may range from 1,000 lb for a personnel access ramp to 50 tons or more for a ship repair pier, and they usually are applied to a single beam. Similarly, uniform live loads may be applied to deck systems parallel to and midway between beams.

Bulk cargo densities range from 45 to 50 ft^3/l.t. for dry bulk commodities such as grain, to 25 ft^3/l.t. or less for ores such as coal, and to 10.5 ft^3/l.t. for iron ore. Grains,

Table 4-5. Unit Weights and Angles of Repose for Selected Dry Bulk Materials

Bulk Material	Unit Weight (lb/ft^3)	Angle of Repose (degrees)
Ores		
Aluminum (bauxite)	83	28 dry–49 at 8% moisture
Copper (copper pyrite)	160	38–45
Iron (limonite)	165	40
Lead (galena)	160–173	35–40
Manganese (manganite)	125	40
Tin (cassiterite)	102–124	35–38
Building materials		
Cement (Portland)	84–100	24–30
Cement (clinker)	88	33
Clay	106–138	15–40
Gypsum	100	40
Natural aggregates	80–100	30–40
Sand and gravel	100–125	25–40
Fuels		
Coal (anthracite)	60–70	24–30
Coal (bituminous)	50–65	32–44
Coke	38	40
Foodstuffs		
Cereal	32–48	40
Flour	38	40
Grains (small; wheat, corn, oats, rice)	44–62	23–37
Salt	56	45
Soybeans, peas	50–60	23
Sugar (granular)	63	35
Miscellaneous		
Scrap iron	63–100	35

Source: This table compiled in part and adapted from ACI (1975) and BSI (2000).

salt, fertilizer, sand, coal, iron ore, and so on may be stored temporarily in piles on the wharf deck. Unless stored in bins or hoppers, the total deck load is limited by the material's natural angle of repose and the width of the pier or wharf apron. Table 4-5 gives typical dry bulk unit weights and angles of repose for common bulk materials.

For pier widths of 50 to 100 ft, the peak load can reach 2,000 to 3,000 lb/ft^2 for materials such as sand and coal piled 20 to 30 ft high. Areas where certain ores are stockpiled may be subject to loads of 4,000 to 6,000 lb/ft^2. The use of low retaining walls can further increase stockpile heights. Saturated unit weights should be considered as appropriate for uncovered materials. Stockpiling of such materials on wharf decks is not normally done, but the surcharge load on the earth behind the wharf and hence the lateral earth pressure on the wharf bulkhead or quay wall can be extremely high, and the overall slope stability as well as bulkhead pressures must be checked carefully (see Chapter 8).

Liquid bulk cargoes are typically handled by hoses and pipelines and exert local concentrated loads at pipe supports, bends, hose towers, and manifold connections. The unit weight of crude oil varies with its source and temperature. The unit weight of crude oil at standard atmospheric pressure and temperature of 15°C (59°F) (STP) ranges from around 49 to 57 lb/ft^3. Fuel oil, lube oil, and heating oil are in the range of 56 to 57 lb/ft^3. Kerosene is around 51 lb/ft^3, gasoline is from 44 to 46 lb/ft^3, and soybean and cottonseed oil are about 58 lb/ft^3, to give a few examples of common liquid cargoes. The designer should always take care to verify particular cargo properties over the expected range of operating temperatures. Design requirements for marine oil terminals are included in the State of California Building Standards Code, *Marine Oil Terminals Engineering and Maintenance Standards* (MOTEMS 2011).

4.2 Vehicular and Mobile Equipment Loads

Contemporary tire-mounted and crawler track-mounted equipment often governs the design of the deck system. The designer should investigate any special equipment that will be used at an early stage with the owner's operation personnel and should verify specific equipment characteristics with the manufacturer. Turning radii and operating limits must be verified, in addition to wheel loads. Manufacturers' general brochures can be misleading in regard to deck loads. For example, the average tread pressures often listed for crawler-type cranes may be inadequate for deck system design because crawler cranes typically have nearly triangular load profiles beneath their tracks, with load peaks that may be up to two times or more the average bearing pressure, as described in the following discussion. The distribution of wheel loads and other concentrated loads to deck system elements is addressed in Section 7.3.

Rubber-Tired Vehicles and Equipment

Most cargo-handling facilities are designed to accommodate a standard highway tractor-trailer truck designated HS-20-44 class loading by AASHTO (2002), as illustrated in Fig. 4-1. The load represents a 20-ton tractor truck with 20% of the weight on the steering axle and 80% on the trailer axle, with the trailing axle having an equal 80%, giving the standard HS-20 truck a gross vehicle weight (GVW) of 36 tons. Other truck load classes may also be designated, such as HS-15, another standard design truck designation, or a greater load such as HS-25, which has a GVW of 45 tons that may be more appropriate for many contemporary marine terminals. As an alternate load case, pier decks also should be checked for a military truck loading consisting of two 24,000-lb axles spaced 4 ft apart. Note that the current AASHTO (2014) LRFD specifications use a notional highway load designated HL-93 based on probabilistic superposition of truck and lane loads to give highway shears and moments of heavier

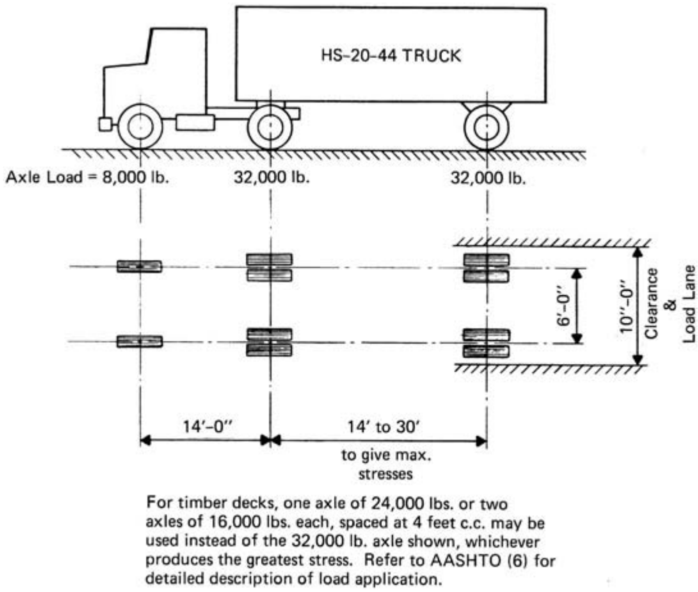


Fig. 4-1. Standard highway tractor-trailer truck wheel loads

Source: Adapted from AASHTO (2014)

trucks. POLB (2012) requires designing for the HL-93 truck loading times a factor of 1.25 plus 10% impact. Lane loads need not be considered. However, in general, considering the short spans and lack of specific travel lanes on piers and wharves, the standard HS-20 design truck is generally acceptable for pier design. An exception may be long-span approach trestles that more resemble highway bridges. Most states issue special oversize vehicle permits, usually with a gross weight upper limit of around 80,000 lb for multiple-axle vehicles. Single and steering axles usually are limited to 22,400 lb and tandem axles to 36,000 lb. Such vehicles, along with other special cargo-handling equipment, should be considered in the design of major cargo facilities. Tractor-trailer trucks typically have a turning radius on the order of 40 to 45 ft. Additional information on turning room for various vehicle types can be found in AASHTO (2001). Impact factors applied to vehicle wheel loads on piers and wharves are usually less than for highway bridges; typically, a 15% increase is applied (DOD 2005), consistent with slower speeds and restricted travel. In addition to impact, longitudinal forces caused by traction and braking may need to be taken into consideration, although in most cases they are small compared with the horizontal forces a pier or wharf is normally designed to accommodate. AASHTO specifications are useful as a design guide for general vehicular loads; for their application and distribution, refer to Section 7.3. Other truck-type vehicles that may need to be accommodated include terminal tractors for moving containers within a container terminal, fire trucks and emergency response vehicles, and possibly fuel trucks, all of which should be identified early in the design stage.

AASHTO (2002, 2014) specifies a tire contact area of 20 in. width and 10 in. length over which the tire pressure can be applied for the design trucks. For other vehicles, the contact pressure and area must be determined by the design engineer. The AASHTO specification commentary gives some guidance for this. One approach to estimating contact area for rubber-tired equipment is to obtain tire size and inflation pressure and maximum wheel loads from the equipment manufacturer. Maximum tire inflation pressure for most heavy equipment is typically in the range of 80 to 125 psi. Mobile cranes may increase the tire pressure on the order of 10% for lifting on rubber lifts. For solid and/or cushion-type tires, such as may be found on some forklift trucks, contact pressure can be on the order of 180 to 250 psi.

Table 4-6, from Sembler (1974), presents some generalized data for various types of rubber-tired equipment that may be found in a contemporary container-handling port. Individual dual tire wheel loads of up to around 100,000 lb are possible. Forklift

Table 4-6. Material Handling Equipment Wheel Loads

Identification Code		Capacity[a]	Drive Axle Load (lb)	No. of Wheels and Wheel Loads (lb)	Design Wheel (lb)
Highway truck forklifts		H-20-44	—	16,000	16,000
	A	30 L.T.	185,000	4/46,150	92,500[b]
	B	40 S.T.	180,000	4/45,000	90,000[b]
	C	30 L.T.	159,000	4/39,800	79,600[b]
	A	20 L.T.	152,000	4/38,000	76,000[b]
	C	20 L.T.	127,000	4/31,750	63,500[b]
	D	20 S.T. at 30 in.	84,000	4/21,000	42,000[b]
	E	15 S.T. at 48 in.	82,700	4/20,700	41,450[b]
	F	15 S.T. at 48 in.	79,600	4/20,000	40,000[b]
	G	15 S.T. at 48 in.	78,500	4/19,700	39,300[b]
	H	7.5 S.T. at 48 in.	48,700	4/12,100	24,400[b]
	J	7.5 S.T. at 48 in.	43,200	4/10,800	21,600[b]
Straddle carriers					
	K	30 L.T.	—	4/27,000	27,000
	L	40 S.T.	—	4/26,000	26,000
	M	20 L.T.	—	4/18,000	18,000
	N	20 L.T.	—	6/13,000	13,000
Transtainers					
Maximum load	40 L.T.	—	4/100,000	100,000	
Maximum load	30 L.T.	—	4/90,800	90,800	
Average working load	25 L.T.	—	4/84,500	84,500	
Empty	0	—	5/60,000	—	

[a]L.T. = long tons; S.T. = short tons.
[b]With dual wheels closer together, effect on pavement would probably be sum of two wheels.
Source: Sembler (1974).

trucks (FLTs) are common in ports and generally have lifting capacities in the range of 5 to 30 l.t., up to 40 tons, with typical wheelbases of 8 to 10 ft, spacings of 6 to 8 ft, and a corresponding turning radius of 13 to 16 ft. A pier designed for an HS-20-44 truck loading usually checks out safely for up to an 8- to 12-ton capacity forklift truck. The U.S. Navy (DOD 2005) recommends that general cargo piers be designed for a 20-ton forklift truck that has almost 50,000 lb for each front dual tire load. Container terminals should be designed to accommodate the largest forklift equipment in use, typically at least 30 and up to about 50 metric ton capacity, with front axle loads on the order of 200,000 lb or more. The Port of Long Beach (POLB 2012) requires designing for container-handling equipment wheel loads with four front wheel loads of 67.25 kips each (269-kip axle load) on a 12.25-ft wheelbase plus 10% impact. Wheel loads are to be distributed in accordance with AASHTO (2002, 2014). A rough rule of thumb for forklift trucks is that the front axle load is typically slightly more than twice the rated capacity. Note that the rated capacity is based upon a given distance to load center so that axle loads based upon rated capacities can easily be exceeded and are typically around 90% to 95% of the total weight of machine plus load. Variations of forklift trucks for handling and stacking containers include reach stackers and full and empty container handler vehicles. Full container handlers are capable of stacking up to six standard containers high. Container terminals are usually also equipped with straddle carriers and transtainers, which have wheel loads on the order of 30,000 lb, up to 100,000 lb or more, respectively. A typical container yard straddle carrier is shown in Fig. 4-2. The largest straddle carriers are capable of stacking up to one over five containers. *Transtainers* are actually rubber-tired or rail-mounted gantry cranes that can span several rows of containers and stack up to six high or one over five containers. Mobile straddle lifts used to haul and transport small craft and other vessels are discussed in Section 10.5.

Fig. 4-2. Container-handling straddle carrier

Fig. 4-3. Heavy load transporter in use at ship building yard

Source: Photo courtesy of KAMAG Corporation of America

Load transporters, often found in shipyards, as shown in Fig. 4-3, are used to move very heavy loads and large loads, such as ship modules and power plant units, and can be custom manufactured to carry loads of several hundred tons. Individual tire loads are generally similar to tractor-trailer truck tire loads. Wheel configurations and load limits must be investigated on an individual case basis.

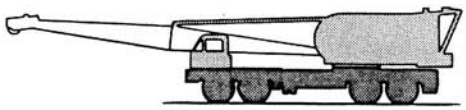
Other types of rubber-tired equipment include scrapers, heavy-duty dump trucks, and front-end loaders such as are commonly used in bulk material-handling facilities. These vehicles typically have large "balloon" tires with 50,000-lb typical wheel load capacity. Aerial lift mobile elevating personnel platforms are often used in dry docks and shipyards and have relatively light wheel loads. Wheel loads and spacing and other operational criteria of any specialized equipment should always be verified from manufacturers' data. Automobiles present relatively light loads. The floors of parking garages, for example, are typically designed for equivalent uniform live loads on the order of 50 to 75 lb/ft^2. The decks of car carrier and other Ro/Ro-type piers should be designed for much higher uniform live loads, however, as well as localized loads of the ships' ramps.

In general, a horizontal force of 10% of the wheel loads of all rubber-tired equipment should be applied longitudinally at deck level to account for traction and braking forces. An additional lateral force of 10% should be considered for truck cranes to account for slewing forces and operating wind loads. In general, a vertical impact factor of 15% should be applied to the wheel loads of all traveling equipment. Impact factors apply to all deck system structural elements but not to the piles themselves.

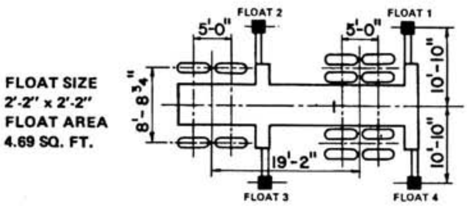
Mobile Cranes

Mobile cranes include rubber-tired truck cranes (TCs); rough-terrain cranes (RTCs); all-terrain cranes (ATCs); larger mobile harbor cranes with many wheels, which generally lift on extended outrigger floats; and crawler track cranes, which may have adjustable track widths. Crane booms are typically of either the truss-like lattice-type typical of TCs and crawler cranes or tubular steel "telescoping cantilever"-type typical on RTCs and ATCs. Rated lifting capacities are generally controlled by tipping moment rather than the structural strength of the crane and are generally on the order of 85% of overturning for lifting on floats and for crawler cranes and around 75% of overturning moment for lifting on rubber lifts. *Timber cribbing* is often used to help distribute heavy float loads, and special oversize floats or spreader beams may be required to distribute heavy loads under certain circumstances. Detailed description of mobile cranes, including loads acting on and exerted by them, can be found in the comprehensive text by Shapiro and Shapiro (2011).

Truck cranes are a versatile and common type of lifting equipment, ranging from 10 to 15 tons up to around 300 tons rated lifting capacity. The larger sizes can be used to lift containers, and at 300 ton capacity can lift a 30 ton container at up to a 100-ft radius. During such a lift, the loads under the outrigger floats are extremely high and must be properly distributed to the deck structure and foundation via mats or special floats. Such equipment preferably is used in solidly filled wharves or quays; otherwise, for economy of deck design, they may need to be restricted to designated heavy lift areas. Individual wheel loads while traveling are similar to trailer truck loadings, but the total vehicle weight is much greater for the larger truck cranes. The U.S. Navy (DOD 2005) requires that all general berthing piers be designed to accommodate a minimum 70-ton capacity truck crane, and outfitting and repair piers must accommodate a 140-ton truck crane. Maximum outrigger float loads for over-the-corner capacity lifts are on the order of 150,000 lb for a 70-ton crane and 230,000 lb for a 140-ton crane. Float loads are for maximum rated capacity lifts with near-vertical boom angle and minimum load radius and decrease with greater load radius, dropping off to around 70% of maximum at some distance beyond which it remains constant as the rated load decreases to minimum. Truck cranes in the 50 to 140-ton capacity range typically have wheelbases between 18 and 20 ft, with about 8 to 9 ft of track width and around 20 ft center to center for floats in the lifting position. Fig. 4-4 gives wheel and float loads for a representative 140-ton capacity truck crane. Additional data on truck crane loadings and general dimensions can be found in BSI (2013), PIANC (1987a), and the various manufacturers' product literature.



140-TON TRUCK CRANE



1. DATA SHOWN FOR OUTRIGGER FLOAT LOADS ARE FOR OVER SIDE AND OVER REAR LIFTS. FOR OVER FRONT LIFTS, A FRONT BUMPER FLOAT IS REQUIRED.
2. BOOM IS OVER THE CORNER FOR WHICH FLOAT LOAD IS GIVEN.
3. FOR EQUAL RADII, RATED LOADS VARY ACCORDING TO BOOM LENGTH.

CRANE TRAVELING – RUBBER TIRE WHEEL LOADS (LBS)

TIRES NO.	TIRES SIZE	BOOM LENGTH (FT.)	TOTAL WEIGHT (LBS.)	BOOM OVER FRONT: EACH FRONT SINGLE TIRE	BOOM OVER FRONT: EACH REAR DUAL TIRE	BOOM OVER REAR: EACH FRONT SINGLE TIRE	BOOM OVER REAR: EACH REAR DUAL TIRE
12	14.00-24	50	168,821	6,711	35,494	13,759	28,447

OUTRIGGER FLOAT LOADS (LBS)

RATED LOAD (LBS.)	RADIUS (FT.)	BOOM LENGTH (FT.)	FLOAT NUMBER (1)	(2)	(3)	(4)
280,000	12	50	172,500	225,500	233,500	164,000
138,360	25		172,500	225,500	233,500	164,000
114,400	30		163,900	214,200	221,800	155,800
74,900	40		146,600	191,700	198,500	139,400
55,100	50	50	133,700	174,800	181,000	127,100
43,400	60	60	131,100	171,400	177,500	124,600
35,100	70	70				
29,200	80	80				
25,200	90	90				
21,400	100	100				
18,500	110	110				
15,900	120	120				
14,000	130	130				
12,200	140	140				
10,500	150	150				
9,050	160	160				
7,700	170	170				
6,650	180	180				
5,550	190	190				
4,500	200	200				
4,300	200	210				
3,500	200	240				
2,650	200	270	131,100	171,500	177,500	124,600

Fig. 4-4. Wheel and float loads for typical 140-ton truck crane

Source: NAVFAC (1980)

Rough-terrain cranes (RTCs) are suited to off-road service and are commonly found at port facilities, where they may be involved in "pick and carry" operations or special lifting other than cargo handling. They range in capacity from around 30 to 120 tons, have only four wheels, and are lighter and more versatile than conventional truck cranes. The center of rotation of the boom base is typically at or near the center of the area of the extended floats, and the maximum float load for an over-the-corner lift can be estimated as approximately 70% to 90% of the combined GVW plus lifted load. For an over-the-side lift, this load would be shared by two floats (Shapiro and Shapiro 2011). *All-terrain cranes* (ATCs) offer the off-road capability of RTCs but may have as many as nine axles and rated capacities up to 1,200 tons. For these larger machines, maximum wheel and float loads must be obtained from the manufacturer.

Mobile harbor cranes (MHCs), such as that depicted in Fig. 4-5, are available with capacities up to around 200 metric tons at reaches of up to 20 m. The capacity drops off rapidly with increasing reach to around 30% of capacity at the maximum reach, which may be up to around 50 m for the largest MHCs. They can be configured for various types of cargo handling, including containers, general cargo, and various bulk and special materials. They are not suitable for road travel but offer versatility for temporary operations and existing facilities, where retrofitting of crane rail tracks may be problematic, or new facilities, where design of fixed rails and power supply infrastructure may be more expensive and offer less mobility than an autonomous MHC operation. Deck load-bearing pressures can be significantly reduced by the use of oversize floats or corner pads.

Crawler cranes mounted on steel treads or tracks are often used on piers and wharves. Such equipment should not be allowed to travel directly over concrete or timber decks; instead, timber mats are usually laid down in the path of the tracks to protect the deck surface and aid in load distribution. Crawler cranes are usually in the same capacity range as truck cranes: 20 to 300 tons. The load distribution under their tracks during lifting and even when idle varies with boom positions from trapezoidal to triangular, resulting in high localized loads. Fig. 4-6 shows a schematic of ground pressure distributions with various boom and load conditions, as well as crawler crane important dimensions. An impact factor on the order of 25% should be applied to the total vertical load to account for possible lift off of the crane track. Peak track pressures can exceed 100 psi (14.4 kip/ft^2) at the peak of the triangular load distribution. Track widths are usually on the order of 36 to 54 in., with a track base of 12 to 16 ft. The axis of rotation of a typical crawler crane usually passes through the centroid of the track-bearing surfaces; except for larger machines, the axis is more toward the rear of the track's centroid. The track-bearing length is usually taken as the center-to-center distance between the drive sprocket and idler sprocket, as shown in Fig. 4-6.

For virtually all traveling cranes or other boomed lifting equipment, the actual lifting capacity drops off rapidly with the boom angle and the distance from the center of rotation. Therefore, the rated capacity of such equipment generally is much larger than the nominal loads to be handled. The peak ground pressure under a track, outrigger float, wheel, and so on usually occurs for an over-the-corner lift. The manufacturer's load/capacity charts are based upon rated safe lift capacities versus reach and usually do not include allowance for wind or impact. Therefore, when the proposed use of such equipment is critical to the design, the resulting loads

Fig. 4-5. Mobile harbor crane handling containership cargo. This crane has a capacity of 125 metric tons at 20 m reach and has seven axles with 28 wheels

Source: Photo courtesy of Terex Port Solutions

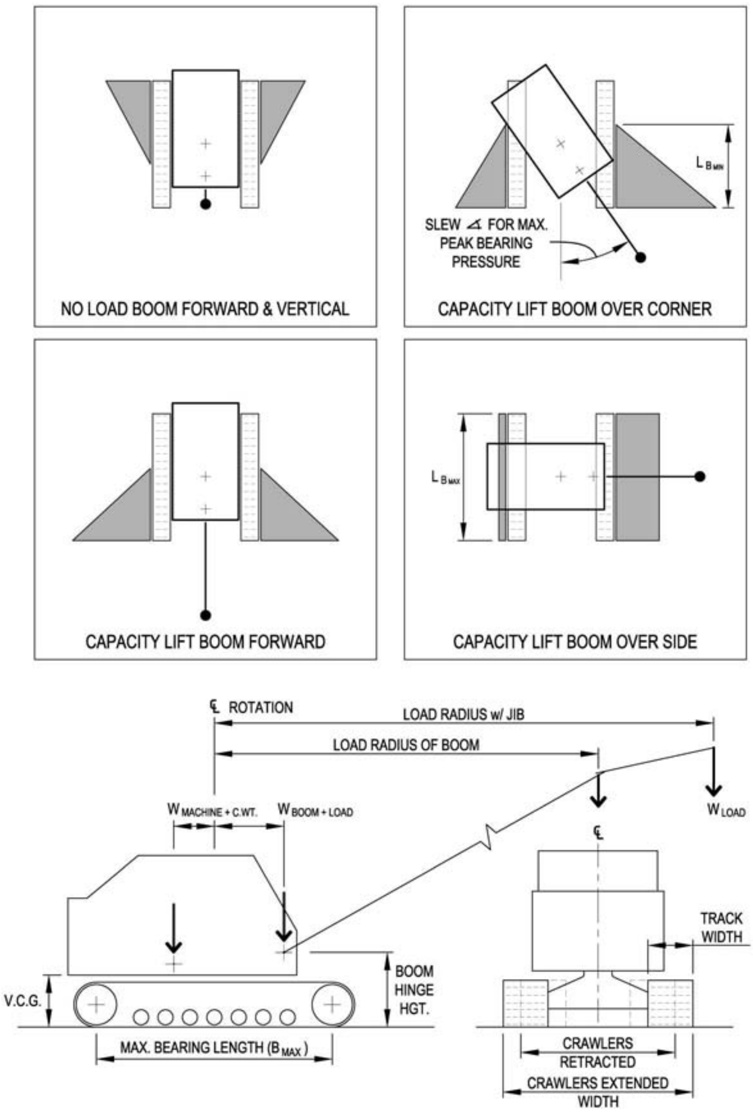


Fig. 4-6. Crawler crane track load distribution and definition sketch

must be examined carefully. More rigorous treatment of mobile crane features and loads can be found in the text by Shapiro and Shapiro (2011).

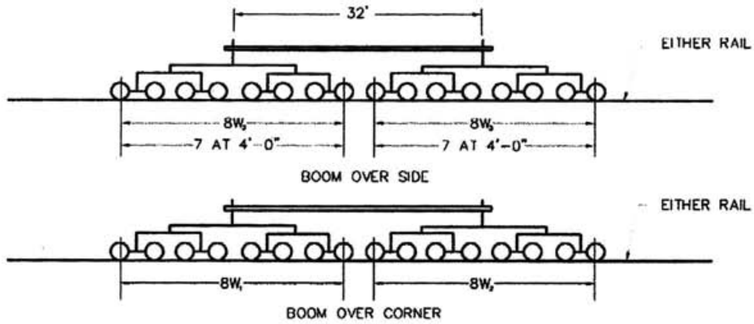
4.3 Rail-Mounted and Material-Handling Equipment

Rail-mounted equipment commonly seen on piers and wharves includes railroad trains, portal-type revolving cranes, gantry types, container cranes of various configurations, and a wide variety of specialized ship loaders, such as those used in bulk handling. Cranes and loaders may alternatively be mounted in a fixed position.

Railroad loadings are given by the American Railway Engineering and Maintenance-of-Way Association (AREMA) in its *Manual for Railway Engineering* (2014). The standard load case is a "Coopers Train," consisting of two locomotives in tandem followed by an infinite number of loaded freight cars. In E-80 loading, such as required by POLB (2012), the string of freight cars is represented by an equivalent linear load (on two rails) of 8,000 lb/ft. Other classes of loading are in direct proportion: Class E-70 has a load of 7,000 lb/ft, and so on. For most piers that have relatively short spans, however, the wheel loads of the heaviest locomotive or sometimes freight car usually govern. Considering the increase in car axle loads over the years, AREMA has adopted an alternative load on four axles of 100 kips per axle spaced at 5-, 6-, and 5-ft centers for a total of 11 ft center to center of the end axles. This load may be more appropriate for pier and wharf design, given the typically shorter spans. Locomotives generally range from 45 to 145 tons in weight, usually distributed evenly over a minimum of eight wheels mounted on two trucks. NAVFAC (DOD 2005) uses a standard locomotive weight of 120 tons with a corresponding single wheel load of 15 tons. AREMA (2014) provides additional railroad load data. A locomotive crane with a 40-ton capacity at a 12-ft reach has a maximum corner wheel load on the order of 35 tons. Although impact factors vary with span length according to AREMA specifications, an impact factor of 20% as used by the U.S. Navy should be sufficient for pier and wharf design purposes. The standard rail gauge is 4 ft 8.5 in. center-to-center of the rails. Additional information on railroad loads and design criteria specific to U.S. government and military facilities is provided by USACE (DOD 2004).

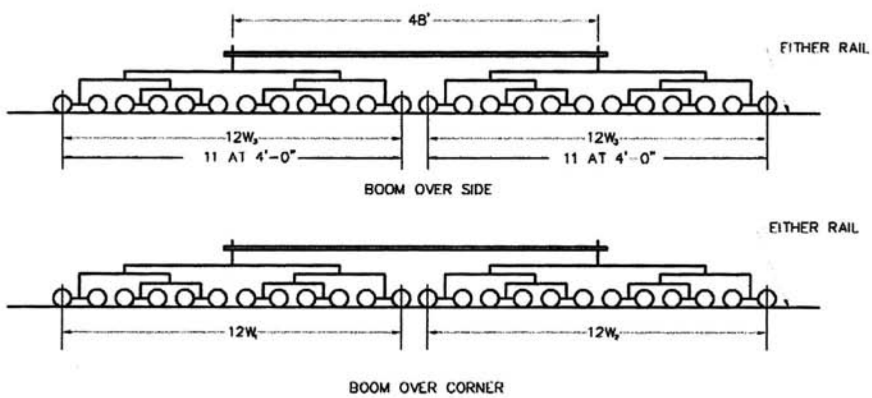
Portal-type revolving cranes, such as the ones shown in Fig. 1-8 are essential features of shipbuilding and repair facilities and also are seen in many general cargo facilities. Such cranes may be equipped with a jib boom to increase their reach and generally are *level-luffing*, allowing the boom angle to be changed without changing the height of the load. Revolving portal cranes have typical gauges from 20 to 50 ft and rated capacities (boom up) on the order of 25 to 100 tons or more, although shipbuilding cranes of 300 tons, with fully revolving capacity, are in use. Fig. 4-7 illustrates some typical portal crane wheel loads for general reference. Wheel loads can be considerably higher for specific installations. Large (especially high) cranes should be checked for peak wheel loads under storm conditions because wind heeling moments may produce higher wheel loads than those experienced during rated lifting. Crane wheel loads are normally provided by the crane manufacturer and should include the expected range of operating conditions and corresponding load combinations. The manufacturer's wheel load data should also include an increase for normal allowable operating wind speeds, which should be verified by the designer. Longitudinal forces caused by traction, braking, and slewing and lurch loads also must be considered. Crane trackage design conditions and loads are discussed in greater detail in Section 7.6.

60 Ton Portal Crane



Main Hook Capacity	Rail Gage (ft)	Boom Over Corner		Boom Over Side
		W_1 (lbs)	W_2 (lbs)	W_3 (lbs)
120,000 lbs. at 90ft.	18	83,000	52,000	72,000
90,000 lbs. at 110 ft.	20	80,000	48,000	70,000
* 40 ft gage cranes have a jib extension and increased boom hinge height. "Boom over side" position is parallel to the rails for the 40 ft gage.	30	73,000	41,000	62,000
	40*	71,000	49,000	64,000

151 Ton Portal Crane



Main Hook Capacity	Rail Gage (ft)	Boom Over Corner		Boom Over Side
		W_1 (lbs)	W_2 (lbs)	W_3 (lbs)
302,000 lbs. at 65ft. 200,000 lbs. at 90 ft.	18	81,000	65,000	76,000
146,000 lbs. at 110 ft. 123,000 lbs. at 120 ft.	40	70,000	58,000	65,000

WHEEL LOADS FOR PORTAL CRANES

Fig. 4-7. Portal crane wheel loads

Source: DOD (2005)

Fixed-base pedestal-type cranes may range in capacity from a few tons for small boat docks to 50 tons or more for shipyard applications. They may also be used at liquid bulk facilities for hose handling or other applications, such as setting gangways. In addition to their normal dead, live, and wind loadings, an impact factor of 100% of the capacity load in the vertical direction and 20% of the capacity load in the horizontal direction should be taken into account for slewing loads of revolving-type cranes.

Traveling gantry cranes or *overhead bridge* cranes that travel on fixed elevated tracks commonly are used in shipyards or stockpiling-type facilities. Such cranes usually have gauges (spans) of 100 ft or more, and spans of 555 ft have been built. Lifting capacities for large shipyard cranes range from 300 to 1,500 tons. Mazurkiewicz (1980) presents an in-depth discussion of shipyard (dry dock) cranes.

Container cranes are conspicuous features of virtually all major seaports. Container cranes may be of various configurations, of which the standard A-frame (see Fig. 1-4) and low-profile, see Fig. 4-8, types are the most basic. These basic types can be modified for longer outreach spans, back-reaches, and/or lift/stacking heights. Most container cranes have a lifting capacity of at least 40 l.t. and gauges of 50 to 150 ft. Some cranes have additional capacity, up to 65 tons, for handling more than one container at a time. The largest container cranes currently in the United States can reach up to 22 containers across a ship's beam, and the largest containerships now servicing some European ports require cranes that reach up to 25 containers across. Other relevant features of container cranes are their outreach, back-reach, and number and spacings of wheel trucks ("bogies") and wheels. Wheel loads are often on the order of 90 to 120 tons at approximately 5-ft spacings, with a total of 10 to 20 wheels per rail. POLB (2012) requires designing crane rail structure for 50 kip/ft. Jordan (1995) provides a discussion of dockside container crane design criteria and a comparative review of available cranes. McCarthy and Vazifdar (2004) address wind

Fig. 4-8. Low-profile-type container crane in use at Port Elizabeth, NJ

Source: Photo courtesy of PACECO, Inc.

loads on dockside container cranes and report that most crane structural failures are caused by inadequate tie-down/stowage pin systems under wind loading. Morris and McCarthy (2001) discuss the effect of jumbo, post-Panamax cranes that have lift capacities of up to 65 m.t. on spreaders to 100 m.t. on the hook.

A wide variety of specialized rail-mounted equipment, "ship loaders" are used in dry bulk cargo-handling terminals, such as the catenary continuous loader shown in Fig. 1-5, which is of the continuous rail traveling type. Radial arm-type loaders, such as shown in Fig. 6-4, pivot about a fixed point that joins a main conveyor system linking ship and shore. Linear-type loaders also connect to a single fixed conveyor point but run on straight rails along the pier face with a shuttling conveyor that rides along with the loading boom as it telescopes with changing position angle. Ship loaders use various types of conveyor systems and clamshells, grab devices, bucket ladders, screw mechanisms, and pneumatic/vacuum systems to pick up and move the bulk materials. The large, complicated superstructures of ship loaders and their modes of operation often require that special attention be given to lateral wind and operational loads. In general, the designer must work closely with the equipment manufacturer during the design stage to verify loadings and other relevant operating criteria. Most often, maximum wheel loads are limited to around 120 to 150 tons because of the practical design capacity of the wheels themselves and local contact stresses in standard rail sections. Section 7.6 proves further discussion of wheel loads and crane rail track design. Ship loaders may also be mounted at fixed locations, thus requiring that the ship be moved along the berth to reach all holds.

Description of all of the numerous types of bulk handling equipment is beyond the scope of this book. Additional general descriptions of marine material-handling equipment can be found in Padron and Paparis (2004). Descriptions of specific equipment installations can be found among the numerous papers published in the proceedings of several ASCE PORTS specialty conferences and the PIANC navigation congresses (see Appendix 2). Additional information on cargo-handling equipment in general can be found in the publications of the International Cargo Handling and Coordination Association (ICHCA) (see Appendixes 2 and 3). In addition to static wind and live loads, certain loading towers and elevated conveyor structures may be subject to dynamic response under wind loadings, as described by Phang (1977).

Liquid bulk cargoes, such as petroleum products and slurries, are transported by pipelines that terminate at manifold locations, with either metal loading arms or hose handling towers, where the connection to the vessel is made. A representative articulated metal loading arm installation is shown in Fig. 4-9. Flexible hoses of greater than about 8-in. diameter require mechanical or hydraulic lifting equipment, such as a pedestal crane, to handle them. Metal loading arms may have pipes up to around 24-in. diameter, and flexible hoses are generally limited to about 10 in. (Padron and Paparis 2004). In addition to the gravity loads of pipes running full, pipelines must be designed for horizontal thrust at bends and elbows. Expansion loops and/or joints must be provided for materials that flow at controlled and variable temperatures. Kemper (1980) provides a useful discussion on the design of

Fig. 4-9. Hose loading tower with 8- and 12-in. flow booms at Finnart Refinery in Scotland

Source: A Ralston photo, courtesy of Woodfield Systems, Ltd.

loading arms for bulk liquid handling, and marine loading arm design and construction standards are provided by OCIMF (1999).

4.4 Port Buildings and Miscellaneous Structures

A common type of building constructed on a pier or wharf, usually on filled-type wharves, is the transit shed. These sheds typically are large single- or sometimes double-story structures used to temporarily store noncontainerized cargoes between shipments. Such structures often have long clear spans and inside clear heights of 16 to 22 ft [the U.S. Navy (DOD 2005) requires 20 ft minimum] for the stacking of palletized cargoes. Prefabricated metal buildings may be used. Single floor levels are preferred, but in passenger and multiuse terminals, two or more floor levels may be used. Building column lines should be made to line up with pier-bent spacing as much as possible. Columns should be protected against collision by forklifts and other cargo-moving equipment, for example, by using concrete jackets around their lower levels. Sometimes, the upper stories or roofs of these buildings are fitted with booms or davits for assisting in cargo handling. Fig. 4-10 shows a prefabricated metal-framed building with fiber-reinforced plastic siding forming a U.S. Coast Guard search and rescue boat house enclosure. The design of these structures is in accordance with usual building codes and standards, with the pier or wharf serving as the building's foundation. Fire protection requirements are normally determined by local building and fire officials. Design guidance for industrial-type buildings, including bridge cranes, is provided by AISC (1993) and AISE (2003). Load and other design criteria for buildings in general can be found in ASCE (2010), ICC (2015), and other applicable building codes with jurisdictional authority and in DOD (2013) for U.S. government and military facilities. Floor loads are suited to the cargoes being stored; refer to Section 4.1. Piers and wharves supporting passenger terminals are subject to more strict building requirements, such as those regarding egress and public safety, and to higher level seismic code requirements, depending upon jurisdiction; see

Fig. 4-10. Prefabricated metal boat house founded on pile-supported concrete finger piers at U.S. Coast Guard Search and Rescue (SAR) Station

Source: Photo courtesy of Appledore Marine Engineering, LLC

Section 4.6. Any habitable structure potentially subject to flooding should be designed to flood-resistant standards, such as FEMA (2000) and ASCE (2014a).

Other fixed structures commonly found on piers include light towers and standards, for which AASHTO (2009b) is helpful; storage tanks or towers for sandblast grit; hoppers and bins for bulk commodities; and support structures for various material-handling equipment. Handrails, ladders, access bridges, gangways, and boarding platforms are discussed in Sections 7.7 and 9.6. Wind loads on buildings and other structures are treated in ASCE (2010) and Simiu and Scanlan (1996). Wind loads on buildings or elevated superstructures may be critical to the design of marine structures, and design wind speeds and appropriate drag coefficients should be selected carefully. Wind loads acting directly on pier or wharf structures themselves usually are not of consequence compared to the usual design loads. However, for lightly framed walkway piers or trestles or single long-span catwalks not subject to other lateral design forces, some provision must be made for lateral resistance. ACI (1995) recommends that a minimum transverse wind load of 300 lb/ft be applied to the design of reinforced concrete bridges, including pedestrian bridges, in lieu of a more detailed analysis. In addition, an uplift/overturning force of 20 lb/ft^2 over the deck area should be applied at the windward quarter point of the structure width.

4.5 Environmental Loads

Environmental loads that may act directly upon marine structures include wind, wave, current, ice, and natural hazards, such as earthquakes and tsunamis. As a general rule, horizontal design loads on vessel berthing structures are governed by vessel berthing and mooring loads, or sometimes, seismic, wind, wave, and current forces acting directly on the structure itself are comparatively small and may be neglected. In some instances, however, they cannot be, including wind loads on buildings, cranes, or elevated superstructures, wave loads on structures at exposed offshore locations, and structures situated in strong currents. Wind loads on moored vessels are addressed in Section 6.5, and the nature of the wind environment and applications to port engineering are introduced in Section 3.4. Wind acting on fixed port structures is treated similarly to other civil engineering structures, as introduced in previous sections of this chapter. Evaluation of hydrodynamic loads on fixed structures is a complex subject whose detailed treatment is beyond the scope of this text. However, the subject is well covered elsewhere, and the following paragraphs are intended to review basic principles and direct the reader to other sources for a more comprehensive treatment.

Wave and Current Loads

Wave loads are dynamic in nature, but for the range of water depths of interest herein, they generally can be represented as equivalent static loads. The nature of wave loadings and the approach to their calculation depend upon the structure or member dimension relative to the wavelength. For example, when the structure width or pile diameter (D) is small with respect to the wavelength (L), the wave motion is relatively unaffected by the presence of the pile, and the resultant force on the pile is caused by water particle velocities (drag forces) and accelerations (inertial forces). When the structure width relative to the wavelength is large enough that it affects the waveform, then the diffraction and scattering of the incident wave must be considered. The resulting forces are primarily inertial in nature in that they primarily depend upon the water particle accelerations. If the structure is very wide with respect to the wavelength, such as a continuous wall, then the incident wave is reflected and forces are treated as a rise in hydrostatic pressure head.

The generally accepted ranges of application of wave force calculations, in terms of the structure or member width-to-wavelength ratio (D/L), are as follows:

- For $D/L \leq 0.2$, drag and inertia forces dominate; use the Morison equation.
- For $D/L > 0.2$, diffraction effects become increasingly important; use diffraction theory.
- For $D/L > 1.0$, pure reflection conditions exist; treat the structure as a seawall.

For pile-supported structures with $D/L \leq 0.2$, loads on individual piles are computed by the Morison equation (Morison 1950), where the total force (F) per unit length of pile is the sum of the drag (F_D) and inertial (F_I) components, as given by

$$F = F_D + F_I = C_D \frac{\gamma}{2g} DU|U| + C_M \frac{\gamma \pi D^2}{4g} \frac{\partial U}{\partial t} \tag{4-1}$$

where

γ = unit weight of water,
g = acceleration of gravity,
U = horizontal water particle velocity component,
D = pile diameter, and
C_D and C_M are the drag and inertia force coefficients, respectively.

For cylindrical piles, C_D is dependent upon the Reynolds number (N_R) (see Section 6.7) of flow and usually ranges between 0.6 and 1.2, and C_M is usually within the range of 1.3 to 2.0. For sharp-edged square piles and H-piles, C_D and C_M are on the order of 2.0 and 2.5, respectively. For a more rigorous treatment of hydrodynamic loads on fixed structures, including selection of hydrodynamic coefficients and general design guidance, refer to API (2014), McConnell et al. (2004), USACE (2006), and DNV (2010). Due consideration should be given to the effects of fouling in increasing projected area and surface roughness. To obtain the total instantaneous force, the water particle velocities and accelerations must be integrated over the water depth (d), and to find the total maximum force, F_D and F_I must be added

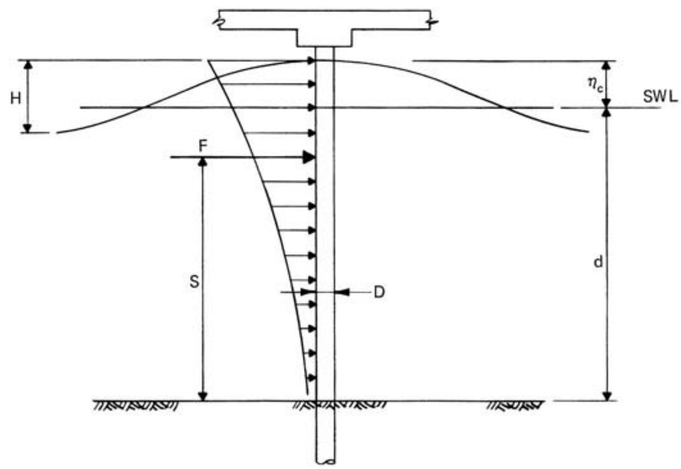


Fig. 4-11. Pile wave force definition sketch

vectorially. Refer to the wave force definition sketch (Fig. 4-11), where η is the wave crest elevation, SWL is the (wave-free) still water level, and the other symbols have been previously defined. The maximum drag force occurs at the wave crest position, and the maximum inertial force occurs at about $L/4$ ahead of the crest. In general, for shallow water and steep waves, F_D predominates, and in deep water and for long, low waves, F_I dominates.

The Morison equation can be rewritten in terms of the incident wave height (H) to find the total force (F_T) on the pile as the vector sum of the drag and inertial components:

$$F_T = \frac{\gamma}{2g} C_D K_D D H^2 \overrightarrow{+} \frac{\gamma}{2g} C_M K_M D^2 H \tag{4-2}$$

where the over arrow indicates a vector addition since the maximums of the drag and inertial components occur out of phase, and K_D and K_M are drag and inertial force factors representing the integration of water particle velocities and accelerations from the bottom to the free surface, as given by

$$K_D = \frac{1}{H^2} \int_0^{\eta_c} U|U| dz \tag{4-3}$$

where η is the free surface elevation and U is multiplied by the absolute value of U to account for the change in direction of the water particle motion as the waveform passes, and

$$K_M = \frac{\pi}{2H} \int_0^{\eta_s} \frac{\partial U}{\partial t} dz \tag{4-4}$$

where η_s is the free surface elevation such that F_T is a maximum. Both K_D and K_M depend upon the period parameters: d/T^2 and H/T^2, where $T =$ wave period.

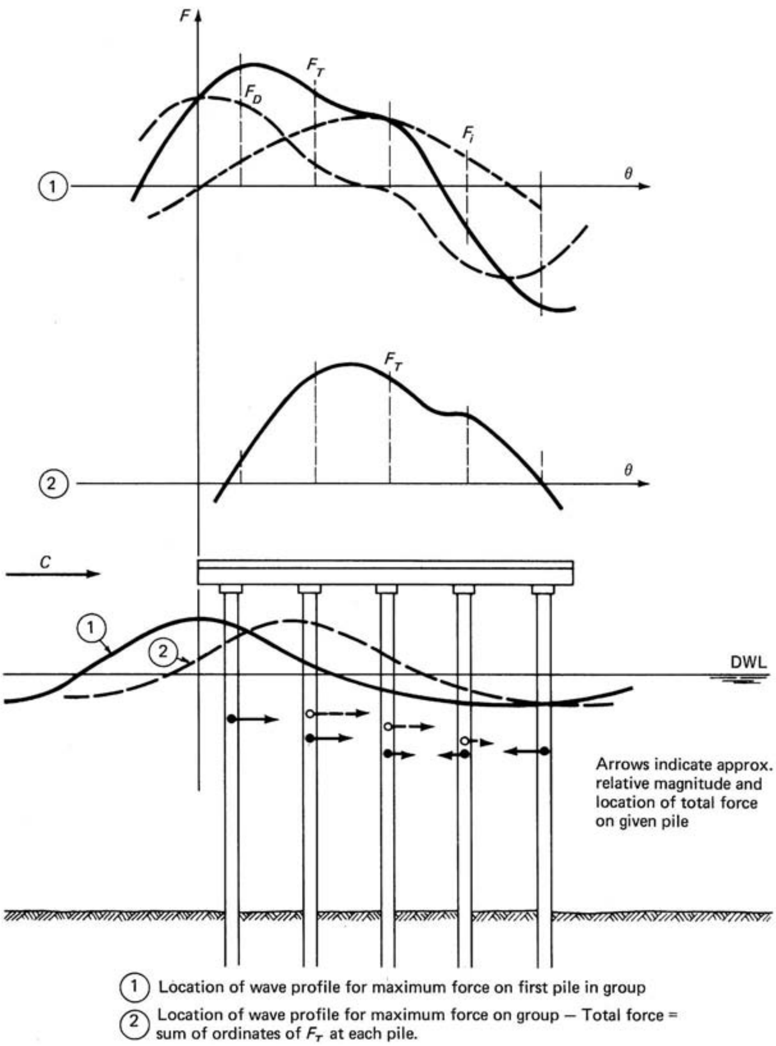


Fig. 4-12. Total wave force on pile structure

Source: Gaythwaite (1981); reproduced with permission

Nondimensional graphs and tables are available to aid in solving the above equations, such as those in Dean (1974) and USACE (2006).

The wave kinematic properties are dependent upon the selection of an appropriate wave theory, which in turn is largely dependent upon the relative water depth at the site. General textbook treatment of wave theories and more in-depth treatment of forces on pile structures can be found in Sarpkaya and Isaacson (1981), Dean and Dalrymple (1991), and Dawson (1983). To find the total overall force on the structure, a succession of wave positions must be plotted, as illustrated in Fig. 4-12.

In the special case of shallow water where the wave breaks on the pile structure, the velocity drag forces generally govern, and the breaking wave force (F_B) per unit length of pile can be found from

$$F_B = \frac{\gamma}{2g} C_B D U_c^2 \tag{4-5}$$

where
C_B = breaking wave drag force coefficient, which can be taken as 1.75 for a cylindrical pile (USACE 1984), and
U_c = wave crest particle velocity.

For shallow water, the wave crest velocity approaches the wave celerity at breaking, as given by

$$U_c = \sqrt{g d_b} \tag{4-6}$$

where d_b is the water depth at breaking, which is approximately equal to the breaker height (H_b). For depth-limited breaking waves, $H_b = 0.78 d_b$, ASCE (2010) gives the following formula for breaking wave forces on piles:

$$F_b = 0.5 \gamma C_D D H_b^2 \tag{4-7}$$

where $C_D = 1.75$ for round piles and 2.25 for square piles.

Wave plus current interactions are very complex because the current modifies the wave kinematics, and steady current velocities must be added vectorially to the wave particle velocities before being squared and after the wave kinematic properties have been modified. API RP-2A (2014) presents a methodology for determining the Doppler shifted "apparent wave period" (T_{app}) to be used to recalculate the wave properties. In addition, the current velocity profile varies with depth, typically with a general boundary layer type profile (see Section 6.6) and must be extended to the wave crest elevation by "stretching" or another accepted method. In general, opposing currents shorten and steepen waves, following currents lengthen and flatten waves, and oblique currents refract waves as well. Swell waves cannot propagate against a current with a speed in knots $\geq 0.75T$, for T in seconds. Under storm conditions, surface currents of 1% to 3% of the sustained wind speed are possible, and when combined with increased tidal currents caused by storm surge effects (see Section 3.4), may result in a significant increase in loads.

For steady uniform currents alone, the drag force per unit length of pile is given by

$$F_C = \frac{\gamma}{2g} C_D D U_{cu}^2 \tag{4-8}$$

Studies of pile groups at varying orientations to the current flow (Ball and Hall 1980) conclude that the total force on the group can be reduced by using fewer piles of larger diameter. Such a reduction in the number of piles should also result in a reduction of siltation and flow field divergence that influences vessel mooring forces.

Transverse lift forces are generated by the periodic shedding of vortices from alternate sides of the pile. The magnitude of the lift force (F_L) is given by

$$F_L = \frac{\gamma}{2g} C_L D U^2 \tag{4-9}$$

where

C_L = the lift force coefficient, which can be taken as approximately=$C_D/3$ for circular cylinders, and

U = the wave and/or steady current water velocity.

The lift force is oscillatory in nature and is usually not of consequence to the overall structure's stability or integrity. However, in strong currents where the vortex shedding frequency is near the natural frequency of pile motion, resonant vortex-induced vibrations (VIVs) may occur. Individual piles are especially susceptible during construction (Khanna and Wood 1979). An example of VIV analysis of an offshore tripod structure is given by Gaythwaite and Mellor (2004). Flow-induced in-line oscillations may also occur when vortex shedding and pile response become locked in. The nondimensional reduced velocity (V_r) as given by

$$V_r = \frac{\bar{v}}{fD} \tag{4-10}$$

where $\bar{v}$ is the mean current velocity and f is the pile natural frequency, which can be used to predict VIV onset as follows. In-line excitations are possible for V_r greater than approximately 1.2 and less than approximately 3.0, and transverse oscillations are possible with V_r greater than approximately 3.5 and less than approximately 7.5, with a peak around 5.5. The amplitude of motion depends upon the mass-damping properties of the pile, and such calculations can be fairly complex. BSI (2013) and Hallam et al. (1978) provide design guidance.

The frequency of vortex shedding (fv) is given by the Strouhal number (S) as

$$N_S = \frac{fvD}{U} \tag{4-11}$$

For cylinders in steady currents and long waves, N_S has an approximate value of 0.2.

The Kuelegan–Carpenter number (N_{KC}) also may be used to predict vortex shedding regimes in unsteady flows and is given as

$$N_{KC} = \frac{U_{\max} T}{D} \tag{4-11}$$

where
U_{max} = average maximum water velocity over the depth, and
T = wave period.

For $N_{KC} < 3.0$, no eddies are formed, and for $N_{KC} > 5.0$, the wake becomes increasingly turbulent. Detailed treatment of flow-induced oscillations of marine structures can be found in Hallam et al. (1978).

In addition to drag, inertia, and lift forces, horizontal members near the surface may be subject to impulsive, vertical wave-slam loads caused by the sudden rise in sea surface elevation. Slam loads are of significance only to local member design and can be estimated from

$$F_S = \frac{\gamma}{2g} C_S D U_V^2 \tag{4-13}$$

where
F_S = vertical slam force per unit length of member,
C_S = slam force coefficient, which has a theoretical value of π for cylindrical members and ranges from about 0.5 to 1.7 (Sarpkaya and Isaacson 1981), and
U_V = vertical water-particle velocity, which, according to small-amplitude wave theory, has a maximum value of $\pi H/T$ in deep water.

Textbook treatment of random sea loads and structure dynamic response can be found in Wilson (2002), Faltinsen (1990), and Chakrabarti (1987).

For large-diameter structures, such as isolated cells or caisson units where $D/L > 0.2$, diffraction theory must be used to calculate wave loads. MacCamy and Fuchs (1954) present a general solution for a bottom-supported, surface-piercing cylinder based on diffraction theory of light waves. The theory has been extended to structures of arbitrary geometry by Garrison et al. (1974) and Hogben and Standing (1975). According to Sarpkaya and Isaacson (1981), the total force is given by an equation of the form

$$F = \frac{\pi}{8}\gamma H D^2 \tanh\left(\frac{2\pi d}{L}\right) C_m \cos(\omega t - \theta) \tag{4-14}$$

where all of the terms are as previously defined for pile structures,
ω = wave circular frequency equal to $\frac{2\pi}{T}$, and
θ = wave phase angle.

Mogridge and Jamieson (1976a, b) present graphical solutions for forces and moments on both square and circular cylinders. The results of physical model studies usually are presented in terms of dimensional analysis in the general form

$$\frac{2F_{max}}{a^2 H \rho g} = f\left(\frac{2\pi a}{L}, \frac{d}{a}\right) \tag{4-15}$$

where a is some characteristic structure dimension; see Section 6.10. Apelt and MacKnight (1976) discuss the results of a particular investigation of caisson structures for a large offshore bulk-coal loading berth. For a theoretical treatment of diffraction theory, the reader is directed to Dean and Dalrymple (1991) and Dawson (1983).

Wave Forces on Walls

When the structure width is longer than the incident wavelength, $D/L > 1.0$, it can be treated as a continuous seawall, where the wall is designed to resist the maximum wave force per unit length along the wall. Fig. 4-13 illustrates reflected/standing wave pressure profiles at walls. Wave forces caused by reflected, nonbreaking waves generally are calculated after the methods of Sainflou and as modified by Miche-Rundgren, which account for partial reflection, as covered in USACE (2006), BSI (2000), and most coastal engineering texts. The ACES computer program (Leenknecht et al. 1992), introduced in Section 3.4, performs these calculations. In general, it is assumed that a standing wave, sometimes referred to as a *clapotis*, equal to twice the height of the incident wave, is formed in front of the structure. The standing-wave profile oscillates about an elevated still water level (SWL), resulting in an increase and a decrease of pressure at the toe of the wall, corresponding to the wave crest and trough positions, as given by

$$P = \gamma d \pm \frac{\gamma H}{\cosh\left(\frac{2\pi d}{L}\right)} \tag{4-16}$$

The total force per unit length of wall is obtained from the area of the net pressure prism. The peak dynamic pressure is at the SWL. The increase, or *setup*, in the water level above the initial SWL is given by

$$h_0 = \frac{\pi H^2}{L} \coth\left(\frac{2\pi d}{L}\right) \tag{4-17}$$

Reflected wave conditions can be assumed when $H < 1.7d$. In shallow water where waves may break against the wall, the Minikin (1963) method was traditionally used but has been mostly abandoned because of the typically very high peak pressures it yields. The method of Goda (2000) is suitable for all cases of wave loading except that of direct-breaking-wave impact pressures, which are not normally considered to be critical to the overall wall stability. The method has been adopted by the USACE in its new *Coastal Engineering Manual* (USACE 2006), which provides instructions for its use in various design applications. The Goda (2000) method was developed for vertical caisson structures on rubble foundations but can be adapted to walls without foundations. The method accounts for direction of wave approach, wave steepness, and bottom slope. Goda recommends a design wave of $H_{\max} = 1.8 \times H_s$, where H_s is the significant wave height. For a normally incident wave, the crest rises to $1.5H_{\max}$ above the SWL and peak pressure occurs at the SWL.

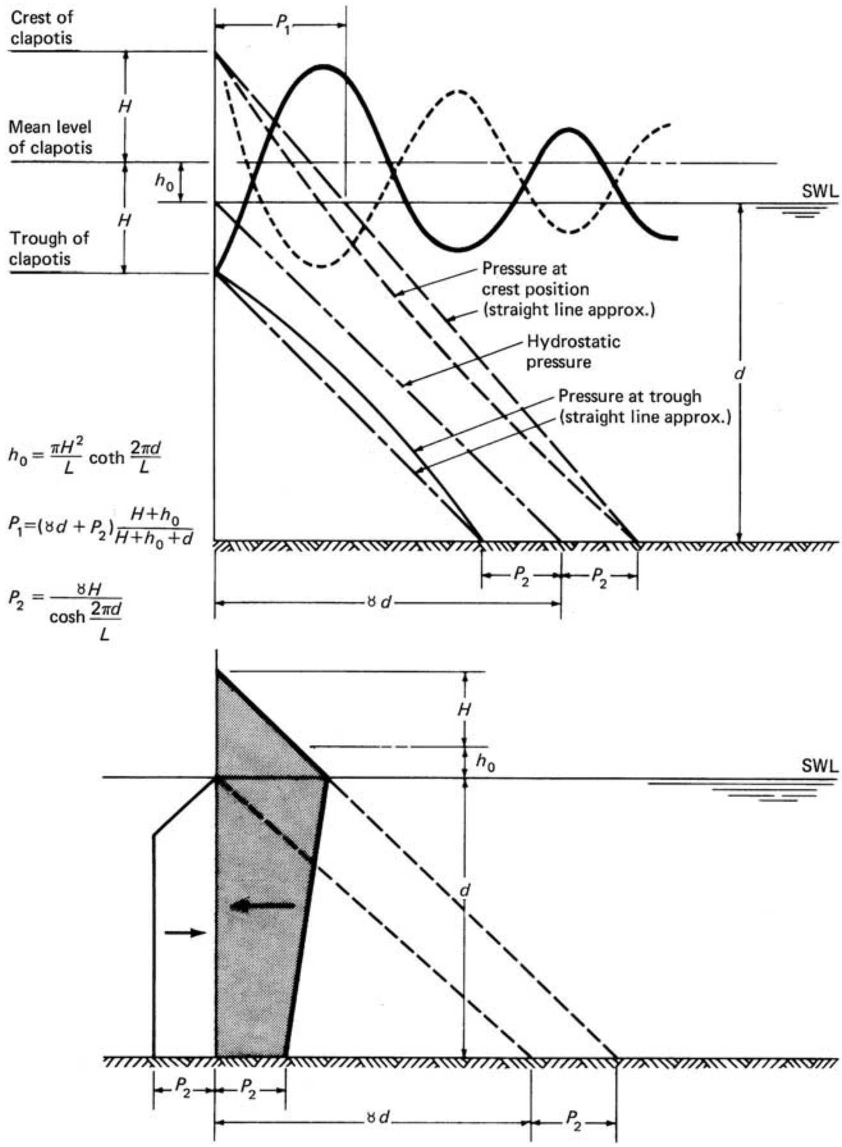


Fig. 4-13. Definition sketch for reflected/standing wave force on wall

A base uplift pressure is also predicted. Goda (2000) provides graphs to calculate the negative seaward-directed force at the wave trough position, which can exceed the crest position forces under certain circumstances. Allsop (2001) presents a rigorous treatment of wave forces on walls, including the importance of impact pressures and negative pressures. Additional useful papers on this subject can be found in ASCE (1995). Most calculation methods result in large total loads when applied over the length of a structure. Battjes (1982) studied the effects of the short-crestedness and directional spreading of waves and found a substantial reduction in total wave force when the structure length equals and exceeds the offshore wavelength. For example, when the structure length and wavelength are equal, the total force is reduced on the order of 25% to nearly 50%, depending upon the directional spreading parameters. For a structure of three or more wavelengths, the total wave force may be reduced by about 67% to 80%.

Wave Uplift on Decks

In general, pier deck elevations should be located above the highest expected wave crest elevation. Where this is not feasible, wave uplift pressures should be considered in the design. Notable early investigations of wave uplift pressures on pier-like structures include the work of El Ghamry (1963), Wang (1967), and French (1979). All of these investigators found uplift pressures characterized by a high localized initial peak pressure of short duration followed by a slowly varying uplift pressure (positive upward, then negative downward) of lower magnitude but of longer duration. The slowly varying pressure is of greatest significance to the overall global uplift force on a structure and its pile foundation, whereas the peak pressures may be critical to local-member design, such as the planks of a timber pier deck. Fig. 4-14 is a wave uplift definition sketch for the following discussion. The most important parameters are the wave crest elevation (η_c) and under-deck clearance above the SWL (s). French noted maximum values of the slow varying pressure head almost equal to the static pressure head at the deck or $\gamma(\eta_c - s)$. El Ghamry placed an upper empirical bound on the peak pressure of five times the incident wave height above the SWL.

Work by Lai and Lee (1989) includes the effect of bottom slope and end reflecting boundary, as would be the case for a marginal wharf-type structure. They developed a numerical model and normalized their data in terms of the total uplift force; F_u=uplift pressure × contact area; and the hydrostatic force: F_s=total weight of water above the platform (the shaded area in Fig. 4-13). They reported maximum values of F_u/F_s in the range of 1.0 to 2.0 for flat bottoms and from 3.0 to 8.0 for sloping bottoms and end walls. It should be noted that all of the cited studies were

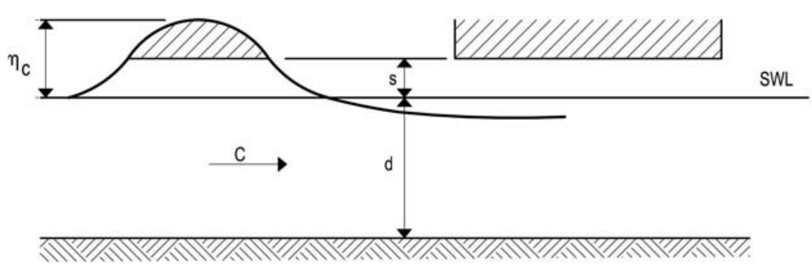


Fig. 4-14. Wave-uplift definition sketch

based on "solitary" waves, whose celerity is controlled by the water depth, with their entire crest height above the SWL. Clearly, there is potential for very high uplift pressures below marginal wharves with steep bottom slopes that are subject to overtopping waves. In addition to the direct wave-induced uplift, a buoyant uplift component can also occur when air is trapped and compressed between beams below the deck slab. Leitass (1979) reports on studies of wave-induced air-pressure-cycling damage to stone slope protections below marginal wharves. Experimental studies have shown a 50% decrease in peak pressures when only a 0.5% area of pressure relief openings relative to deck area were provided. Murali et al. (2009) report on model studies of wave uplift on deck slabs with varying wave steepness in regular and random waves. Mattila et al. (1998) present a case study of a pile-supported cruise ship pier at an exposed location, based on physical and mathematical modeling and literature review. Vertical uplift forces were found to be an order of magnitude higher than the lateral wave forces. Overbeek and Klabbers (2001) present a more empirical case study and report on wave uplift damage to two jetty platforms, and Bea et al. (1999) summarize results of studies of hurricane damage to offshore platform decks in the Gulf of Mexico.

Current design guidance can be found in McConnell et al. (2004), who provide nondimensional graph data for both slow varying and peak impact vertical and horizontal forces based upon physical model tests of a jetty deck, including beam elements and AASHTO (2008) and USDOT (2008) design guidance for highway bridges in the coastal zone. These guidelines also include criteria for determining the maximum wave crest elevation that is critical to deck uplift calculations. A simplified approach to estimating wave uplift and corresponding horizontal forces as developed by Douglas et al. (2006) can be found in FHWA (2008). The slow varying wave uplift force (F_{usv}) can be estimated from;

$$F_{usv} = C_{usv}\gamma(\eta_c - s)A_{vp} \tag{4-18}$$

where

C_{usv} = slow varying uplift pressure coefficient,

γ = unit weight of water, and

A_{vp} = vertical projected area over which the wave acts.

The reasonably accurate value of C_{usv} is 1.0 if all parameters are accurately known; however, for conservative estimates, a value of 2.0 is recommended (USDOT 2008). Peak impact forces of very short duration, which may be important to checking local stresses on utility connections for example, can be four or more times greater than $C_{usv} = 1.0$. Corresponding horizontal forces, the maxima of which may be slightly out of phase, are estimated with an equation similar to Eq. (4-18), except that s is replaced by the vertical distance to the center of the horizontal projected area and additional terms for number and location of below-deck girders are introduced. As this formula is peculiar to typical bridge construction, the reader is referred to USDOT (2008) to determine if it is appropriate to a given pier construction or to the other referenced guidelines when the horizontal force is of concern.

Ice

Ice imposes the following kinds of loads and effects on structures: uplift and pile jacking loads, lateral thrust, impact from drift ice, increased weight and wind profile area caused by ice accumulation, abrasion of timber and concrete in the tidal zone, and expansion forces of ice in captive pockets. Even structures in temperate climates subject to occasional ice covers of, say, 6- to 12-in. thickness or more should be designed to resist nominal ice uplift forces. This is especially true for light-timber pier construction, such as that shown in Fig. 4-15. Typical ice problems encountered in a small-craft harbor are illustrated in Fig. 4-16. The effects of ice on moored vessels are addressed in Section 6.10.

Wortley (1984) presents minimum suggested ice uplift loads on piles of various dimensions and for various ice thicknesses. Wortley's work is based upon a "first crack" analysis, whereby radial cracks extend outward from the center of the pile to a circumferential crack at a distance (a) called the radius of load distribution. Wortley notes that for marina piles in the Great Lakes, this crack typically occurs approximately 6 in. out from the face of the pile, so that for estimating purposes it can be taken as the pile radius plus 6 in. The uplift loads are directly proportional to ice flexural strength. Using Wortley's method and assuming an ice flexural strength of

Fig. 4-15. Light-timber pier encumbered with ice. Note coverage of cross bracing and ice collars on piles

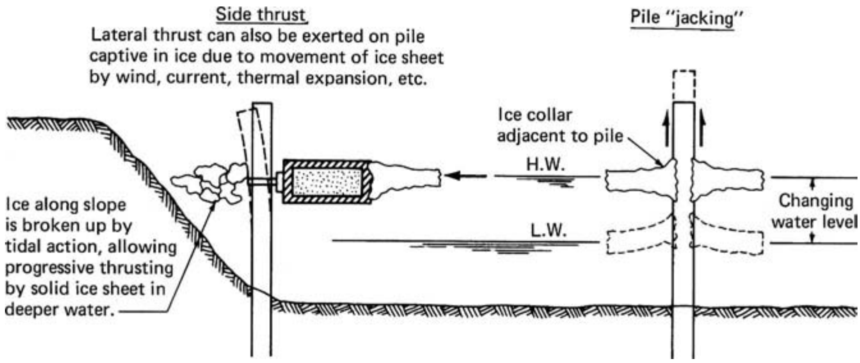


Fig. 4-16. Ice problems in a small-craft harbor

Source: Gaythwaite (1981); reproduced with permission

200 psi, ice uplift loads on a 12-in.-diameter timber pile would range from approximately 8,000 to 30,000 lb for ice thicknesses of 12 and 24 in., respectively. For a 12-in.-diameter steel pile, the uplift forces would be approximately 9,000 and 33,000 lb, respectively. Increasing the pile diameter, however, does not dramatically increase the uplift force. For example, a 24-in.-diameter steel pile in a 24-in.-thick ice sheet would be subject to 37,000 lb uplift, and a 36-in.-diameter pile to 42,000 lb uplift, as compared with 30,000 lb for the 12-in.-diameter pile. Effective pile adhesion values are generally less than 50 psi for timber and steel and generally less than 70 psi for concrete piles.

Lateral thrust caused by moving ice can be extremely high. Dams and structures in confined areas are subject to lateral thrust caused by thermal expansion of ice that can be on the order of 20 to 30 tons per lineal foot. The calculation of lateral ice thrust usually is based upon an equation of the form

$$F_i = C_i f_{ic} A_{ic} \tag{4-19}$$

where

C_i is a coefficient depending upon the structure shape and configuration, the rate of load application, and so on, which normally ranges between 0.3 and 0.7;

f_{ic} is the ice compressive strength, usually assumed to be between 100 psi and 400 psi; and

A_{ic} is the contact area, which is equal to the pile diameter or structure width times the ice thickness.

As can be seen, Eq. (4-19) leaves room for a lot of assumptions. In reality, however, the lateral force of the ice sheet often may be limited by the driving force rather than the compressive strength of the ice, such as wind and current acting on an ice sheet along the coast or within an open bay. In this case, the wind or current velocity and shear stress are the governing parameters. Here, the driving force can be calculated using the well-known drag force equation (see Section 6.3), which for wind reduces to

$$F_{iw} = 0.0034 C_{iw} V^2 A_{is} \tag{4-20}$$

where
F_{iw} = force in pounds,
C_{iw} = wind shear stress drag coefficient, which normally ranges from 0.002 to 0.01,
V = wind velocity in knots, and
A_{is} = surface area of the ice sheet.

For current-driven ice, the equation becomes

$$F_{ic} = C_{ic} U^2 A_{is} \tag{4-21}$$

where
C_{ic} ranges from around 0.01 to 0.1, and
U is in feet-per-second units.

Fig. 4-17 shows the force per unit length of structure for an effective ice sheet length of 1 nautical mile for wind and current action based upon the above relationships. In rivers and narrow channels, the length of ice sheet considered generally does not need to be more than three to five times the river or channel width for an intact ice sheet because it can be assumed that beyond that length, the ice sheet is restrained by shoreline friction. Maximum ice forces often occur during spring breakup when the ice sheet breaks free of the shoreline and often separates into large ice floes.

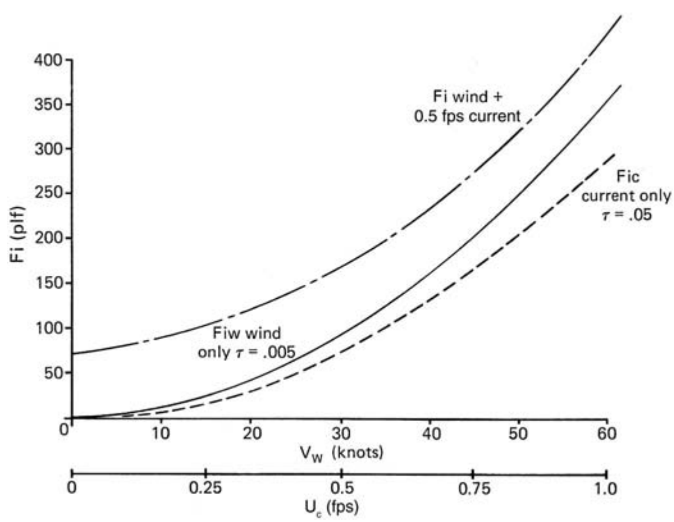


Fig. 4-17. Lateral thrust of ice sheet driven by wind and current

Impact loads caused by drift ice can be even more damaging when borne by strong currents. In order to estimate ice-impact loads, it is necessary first to assume a weight of ice floe and an impact velocity. Wind-driven ice may travel at 1% to 7% of the mean wind speed, but usually it is less than 3%. Ice borne by currents can be assumed to travel at the water surface velocity. Cammaert and Tsinker (1981) studied ice impact forces on marine structures and postulated that approximately one-half of the kinetic energy of an ice floe striking a rigid pier would be dissipated in progressively crushing the ice, and then the piece would rotate and pass by. See Section 6.11 for the effects of ice on moored vessels.

There is a substantial body of literature on designing for ice forces, especially under arctic conditions, such as Caldwell and Crissman (1983), Cammaert and Muggeridge (1988), Eranti and Lee (1986), and Tsinker (1995). The State-of-the-Art Reports by the USACE Cold Regions Engineering Laboratory (CRREL/IAHR 1980–) provide many useful papers on ongoing research and applications. General design guidance can be found in USACE (2002, 2006), and specific design guidance for arctic structures is given by API (2015) and ACI (1991).

4.6 Seismic Loads and Ground Motions

Damage to port and harbor structures caused by earthquakes has been well documented, most notably by Percher (2014), Edge (2013), PIANC (2001a), and Werner (1998). The most severe damage typically occurs in high seismicity zones with soft and liquefiable soils, which generally results in large ground deformations caused by lateral spreading and liquefaction. As a result, most port structures fail because of excessive deformations as distinguishable from the collapse mode of failure more typical of buildings and bridges. Hence, displacement-based design (DBD) methods are generally more desirable, except perhaps for port structures in low seismic hazard zones that are founded on substantial soils and that may be amenable to the somewhat simpler analysis methods of force-based design (FBD). Soil–structure interaction (SSI) is of key importance because of the strong linkage between the soils and foundations in waterfront structures. Structure type is also an important factor in failure modes. For example, gravity-type quay walls, such as caissons, are subject to horizontal displacements and settlements and to tilting, especially when the ratio of width to height is less than around 0.75 (PIANC 2001a), sheet pile quay walls and bulkheads are more susceptible to structural strains and anchor system failures, and cellular-type quay walls are subject to displacements and settlements, as well as structural joint/interlock failures. Pile-supported structures may fail because of soil movements below the deck or to large deck displacements and pile failures, and the use of batter piles in high seismic zones may result in pile connection and local deck system failures, as discussed in Section 7.4. Cranes may suffer derailment, detachment of trucks, and/or pullout of hold-down clamps and anchors and possible overturning and/or buckling of legs. Conveyor systems, boarding platforms, and other deck-mounted equipment may have their own modes of failure.

Earthquake ground motions are typically characterized in terms of a peak ground acceleration (PGA), usually as a percentage of gravity, and may be applied directly to the determination of seismic loads or to develop response spectra or time histories for dynamic analysis. PGAs have been mapped for the United States and its territories by the U.S. Geological Survey and are presented in design codes and standards such as ICC (2015) and ASCE (2010). Specific seismic design guidelines have been developed for piers and wharves. The ASCE/COPRI standard, *Seismic Design of Piers and Wharves* (ASCE 2014b), specifies performance levels associated with the severity of ground motions based upon a statistical return period and three design classifications associated with the consequences of damage or failure. Design criteria are established with regard to structure performance for three basic reference levels. An operating-level earthquake (OLE) is defined as having a 50% probability of exceedance over a structure's life, which corresponds to a 72-year return period over a 50-year design life. A contingency-level earthquake (CLE) is defined as having a 10% probability of exceedance over a structure's life, corresponding to a 475-year return period for a 50-year design life. And a design-level earthquake (DE) is defined to be in accordance with requirements for buildings under ASCE (2010). The DE seismic hazard and performance level is derived from a maximum considered earthquake (MCE), which has a 2,475-year return period. Under an OLE, the structure would continue in operation without structural failure but with some minor reparable damage possible. Under a CLE, some structural damage may occur but without collapse, and it should be controlled and reparable. The DE level is specified by the governing authority where life safety is paramount. Maximum PGAs are taken as $2/3 \times$ MCE. Strain levels for steel, reinforced concrete, and prestressed concrete piles are specified for the OLE, CLE, and DE events. ASCE (2014b) applies only to pile-supported structures and does not apply to piers or wharves that provide general public access, such as passenger and ferry terminals, or to other defined critical facilities. Performance requirements and acceptable damage levels are further defined in the cited guidelines. Design guidance for waterfront retaining-type structures is provided by Ebeling and Morrison (1993) and ASCE (1994). The state of California has taken a leadership role in the development and inclusion of seismic design requirements for marine oil terminals in its building code (MOTEMS 2011). MOTEMS is referenced by DOD (2005) and includes alternative provisions for existing marine terminals, as well as new terminals. The ports of Los Angeles and Long Beach, California, have their own specific seismic design requirements for port structures (POLA 2010, POLB 2012). Other nations subject to high seismic hazard, such as Japan, have their own rigorous seismic standards (OCADI 2008). Johnson and Hardy (2009) provide a history of seismic design codes relevant to piers and wharves in the United States, and a summary of the state of practice and review of existing port guidelines through 2012 is provided by NEHRP (2012).

Because most piers and wharves are designed to resist relatively large lateral forces, they are generally relatively rigid, with natural periods often on the order of 0.5 s or less, although they can be much longer, and thus they offer good resistance to earthquake ground motions when founded on suitable soils in locations with low to moderate earthquake hazard. Design calculations should include the structure dead load only case plus a range of live load conditions, usually between 10% and 50% of the full design live load. Live loads are often ignored for the inertial seismic case at marine oil terminals. In the case of multiple use facilities, the appropriate percent of live load to be added to the seismic mass must be carefully considered. Crane stability and appurtenant structures must also be addressed. Kinematic loading of pile-supported wharves, wherein the soil exerts pressure on the piles as it flows past, is another important consideration. This is an additional force distinct from the inertial loading that must be investigated as to its timing as well as magnitude.

Retaining-type structures with rigid vertical faces may be subjected to an alternate increase and decrease of hydrostatic pressure at the face of the wall during earthquakes associated with an "added mass" of water that resists movement, as illustrated in Fig. 4-18. According to Westergaard (1933), the added mass has a parabolic form that is added to and subtracted from the hydrostatic pressure against the wall, where the pressure at any depth y below the surface is given by

$$p(y) = 7/8\alpha_h\gamma\sqrt{yH} \tag{4-22}$$

where

$p(y)$ = hydrostatic pressure increase/decrease at y below surface,
α_h = horizontal acceleration,
γ = unit weight of water, and
H = water depth at face of wall.

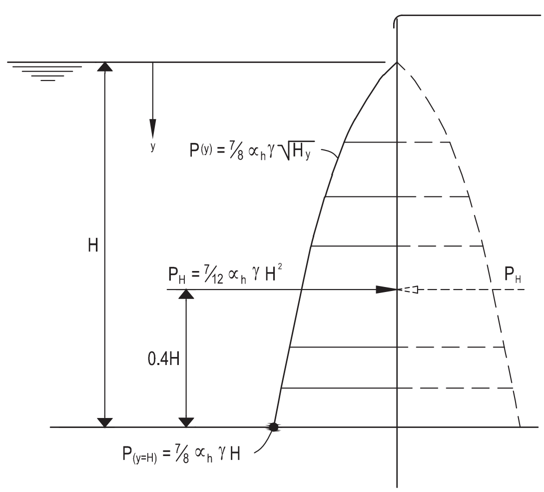


Fig. 4-18. Earthquake-induced hydrostatic pressure on vertical wall caused by added mass of water

The total force at $0.4H$ above the bottom is given by

$$P = 7/12\alpha_h \gamma H^2 \tag{4-23}$$

The variation in soil pressure behind the wall for sandy soils can be determined by the Mononobe–Okabe (M–O) method, which modifies the classical Coulomb theory of soil pressures as modified by the inertia forces caused by ground shaking, as described in ASCE (1994) and in many geotechnical texts.

At locations with soft soils and fills, typical of many estuary harbor sites, and locations with high earthquake risk, seismic design criteria may control, and more elaborate dynamic analysis is required. Soils such as saturated sands, low-plasticity silts, and gravels with little or no fines are especially subject to liquefaction, which could result in catastrophic failures. Fig. 4-19 shows such a catastrophic failure at Kobe Port, Japan, from the Great Hanshin 1995 earthquake.

Methods of analysis depend upon the structure type as well as the level of seismic hazard and subsurface conditions. A simplified quasistatic or response spectrum analysis may be adequate for most structure types at low-risk sites and with suitable foundation conditions. A simplified dynamic analysis, such as pushover or response spectrum analysis, may be applicable to pile-supported structures and a Newmark-type analysis (Newmark 1965) to the under-deck bottom slope below wharves in moderate conditions, whereas a full dynamic analysis considering SSI and consisting of linear or nonlinear finite element (FEM) and/or finite difference (FDM) computer modeling in 2D or 3D may be required at sites with high seismic

Fig. 4-19. Earthquake liquefaction damage to container terminal wharf, Rokko Island, Kobe Port, Japan, January 1995

Source: Photo courtesy of Martin Eskijian, P.E., D.P.E.

hazard and difficult subsurface conditions, and for facilities with high economic value. Seismic analysis of retaining walls and gravity-type structures may be carried out by a variety of methods, as summarized by PIANC (2001a). Seismic design and detailing are addressed further in Section 7.4.

4.7 Tsunami Effects in Ports

Tsunamis are impulsively generated dispersive waves of long length and low steepness. They are typically generated by submarine earthquakes but may also be generated by submarine landslides, volcanic activity, explosions, or other impulsive force mechanisms. They are virtually undetectable at sea with heights typically less than 1 m and long wavelengths associated with periods of a few minutes to an hour or more. They travel at the shallow-water wave speed: $C = \sqrt{gd}$ and increase in height dramatically as they propagate into shallow water, often resulting in very high run-up heights. Tsunamis often manifest themselves in a series of periodic surges that may persist over a period of several hours and may also initiate harbor seiching. They may also or alternatively form bores that may be from a few to many meters high and rush forward at speeds exceeding the shallow-water wave speed. FEMA (2000) recommends a design velocity (U_{ts}) of a tsunami surge propagating over a dry, flat to mildly sloping bed, as given by the following equation:

$$U_{ts} = 2\sqrt{gd} \tag{4-24}$$

where g is the acceleration of gravity and d is the inundation depth. This value should represent an upper bound, and more precise calculation of water velocity caused by tsunami surge at any given location is much more complex. Camfield (1980) provides a comprehensive overview of tsunami engineering.

The North Pacific rim coastlines are especially susceptible to tsunamis. On March 11, 2011, a major tsunami affected the northern Japan coastlines of the Tohoku and Sendai Provinces associated with the great Tohoku Earthquake. Surge heights exceeded 35 m at certain locations, and more than 10,000 vessels were grounded or destroyed, in addition to significant loss of life. The effects on port facilities have been well documented by an ASCE/COPRI field survey team (Percher 2014) and by PIANC (2014). Fig. 4-20 illustrates the severity of the kind of damage that occurred. Port infrastructure damage caused by the Chile earthquake of 2010 has also been documented by an ASCE/COPRI investigation team (Edge 2013). The Great Sumatra earthquake and tsunami of December 2004 caused significant damage to port infrastructure along the southeast India coast and Andaman Islands (Eskijian 2006, Riggs 2007). The ports of Chennai, India, and Port Blair, South Andaman Island, were especially hard hit, with major damage primarily caused by vessels breaking away from their moorings.

Major problems associated with tsunamis within ports and relatively protected harbors are the very strong currents created by the rapidly changing water level;

Fig. 4-20. Vessel of 6,175 DWT "beached" on deck of filled wharf at Kamaishi Port, Suga, Japan, because of the Great Tohoku Tsunami of 2011

Source: Photo courtesy of ASCE/COPRI Port and Harbor Survey Team

surge height under both rising and receding conditions; and impacts by waterborne objects, including moored vessels and other debris. PIANC (2010) presents a detailed overview of tsunami problems in ports, including case histories and recommendations for mitigation measures. Solid fill-type wharves and quays generally fare well, except for some documented scour-related failures, but pile-supported piers and floating structures, including moored vessels, are in much greater jeopardy and should be evaluated for the anticipated currents and high water levels. Headland et al. (2006) describe tsunami effects on moored and maneuvering vessels, which are subject to vertical movements with rising water levels and horizontal forces that can be dynamic and/or quasistatic in nature related to accelerated currents and dynamic forces associated with the leading wave fronts. A comprehensive review of damage to small vessels (generally <5 metric tons) in the 2011 Tohoku event has been well documented by Suppasri et al. (2014). This investigation found that the probability of damage was significantly increased for tsunami wave heights > 2 m and flow velocities >1 m/s. Dykstra and Jin (2006) describe detailed modeling of locally generated tsunami propagation within the ports of Los Angeles and Long Beach, California, and Lynett (2013) describes simulations of tsunami-induced currents in California ports. Ko et al. (2015) describe scale-model testing of tsunami-driven impacts of shipping containers in ports. Tsunami hazard maps that provide contours of expected inundation levels under various tsunami event scenarios are available for most U.S. West Coast and Alaska and Hawaii port areas.

4.8 Other Load Sources and Design Considerations

General

Other load sources also may act on marine structures. Soil pressures and excess pore water pressures caused by hydrostatic pressure heads behind walls and bulkheads are addressed in Chapter 8. Temperature, shrinkage, creep, and support settlements are treated in similar manner as for typical land-based structures, except that the more constant water temperature tends to reduce temperature difference extremes for temperate climates in particular. Buoyancy and installation forces must also be considered. Dynamic amplification of cyclic, random, impulsive, and long-term cyclic loads results in a progressive increase of deflections and a reduction of material strength, such as via fatigue and vibration effects. In general, dynamic effects are not significant where some natural frequency, $f_n = 1/T_n$, of the structure or its elements is less than one-half or greater than three times the forcing frequency. Where dynamic response does occur, the loads are a function of the structure's mass and stiffness properties, and a more detailed analysis is required. This subject is introduced in Section 7.3. Dynamic analysis is not usually required of most nearshore or in-harbor structures, however, because of their great rigidity and ability to resist lateral loads. Impact dynamic load factors are applied to impulsive and moving loads, which may then be treated as static loads. Accidental impacts and the possibility of damage caused by ships with bulbous bows striking the underwater portions of piles or walls must also be considered (PIANC 1990).

In contrast to most land-based structures, marine structures must be designed to resist relatively large transient live loads and lateral forces. Impact and overloading often result in progressive cumulative damage. Therefore, impact resistance and resiliency also should be considered in determining allowable stresses and material requirements. Redundancy and ductility should be provided wherever possible so that the structure cannot fail in a single catastrophic mode. Equivalent lateral load capacities for typical piers and wharves generally range from approximately 500 to 1,000 lb/ft for vessels generally under 2,000 displacement tons (DT), and from 2,000 to 4,000 lb/ft or more for the largest oceangoing vessels. These loads are applied horizontally at deck level and apply only to structures that are continuous over the entire vessel-berth length. The lateral capacity also varies, in particular with vessel type, site exposure, and other factors. In addition to carrying out specific load calculations for a given structure, it is useful for the designer to calculate the equivalent lateral load capacity for comparison with other structures of similar type and function to ensure that an adequate minimum lateral capacity is provided.

Vessel Collision and Impacts

Whereas it is not usual to design marine terminal structures for direct vessel impacts because fendering is normally provided at any expected points of contact (see

Chapter 5), there may be instances where vessel collision, properly termed an *allision* when a moving vessel strikes a fixed object, evaluation of such damage or potential damage is required. The literature on bridge pier protection is especially useful for this. In particular, AASHTO (2009b) and PIANC (2001b), which includes a comparison of design requirements from various nations, provide guidance. According to AASHTO, the head-on static equivalent ship impact force (F_{si}) on a bridge pier, based on experimental data, is given by

$$F_{si} = 8.15V\sqrt{\mathrm{DWT}} \tag{4-25}$$

where
V is the ship impact speed in feet/second,
DWT is the ship's deadweight tonnage in metric tons, and
F_{si} is in kips.

The referenced documents include detailed evaluation of the probability and risk of such things as collision damage, design vessels, and approach conditions, which are beyond the scope of this book.

Marine structures may also be exposed to impacts from various objects and floating debris borne by strong currents such as those associated with storm surges, extreme river runoff, and other flood scenarios such as tsunamis. Debris may pile up against a structure and act as a dam, imposing excess hydrostatic pressures under certain circumstances. Impact forces can be evaluated using various analytical methods, all of which include the mass and velocity of the impacting object and the stiffness of both the moving object and the fixed structure. A simplified method adopted by FEMA (2000) involves an impulse-momentum approach that requires the assumption of a deceleration time (t) for the impacting object to come to a full stop. According to FEMA, the impact force (F_i) is given by

$$F_i = WU/gt \tag{4-26}$$

where
W = object weight,
U = velocity at impact taken to be equal to the current velocity,
g = acceleration of gravity, and
t = deceleration time, assumed to range from a few tenths of a second to 1.0 s.

For concrete structures, t is only 0.1 to a few tenths of second, and for wood structures, t should be taken as 1.0 s. This equation considers only direct, head-on impacts where the object comes to a full stop and does not explicitly include the effects of added mass. If the center of gravity and the center of inertia are not aligned, then the object rotates and possibly passes by the structure, resulting in reduced impact force. Eq. (4-26) also represents the time-averaged impact force and should be multiplied by $\pi/2$ to get the peak force. Loose debris, such as logs in the

Pacific Northwest (Haehnel and Daly 2002) for example, and shipping containers, which can weigh from around 8 kips empty to 67 kips full, and still float, are not unlikely under flood conditions. Ko et al. (2015) present a review of approaches to determining debris impact forces along with their results of scale-model hydraulic tests of shipping container impacts under tsunami currents.

Scour and Propeller Wash

Another source of loading not previously discussed is the strong discharge current produced by a vessel's propeller or bow/stern thrusters, generally referred to as *prop wash*. This is an especially important consideration at ferry and Ro/Ro terminals or other quick-turnaround-type operations, as a source of bottom scour and erosion. At certain locations, it also can be a source of local member loading, such as a current drag force producing bending moments in piles. Fig. 4-21 illustrates the general geometry of the propeller discharge jet or *screw race*. There is a turbulent zone of flow establishment immediately behind the propeller where the maximum initial discharge velocity U_0 is confined to a small area slightly less than the propeller diameter (D_P). Further beyond, the flow spreads out and becomes more uniform, with a velocity $U_{\max}$ along the central axis.

There are several empirical formulas for predicting wash velocities at a given distance behind and below the propeller. The simplest formula requiring the least information about the vessel propulsion system, used by Berg and Cederwall (1981) for the maximum velocity in the turbulent zone, is

$$U_0 = 0.95 n D_P \tag{4-27}$$

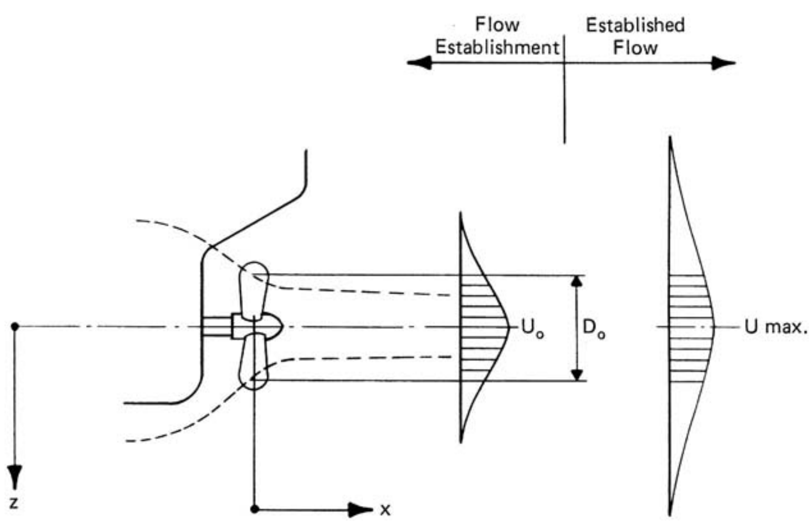


Fig. 4-21. Propeller wash definition sketch

where
U_0 = initial discharge velocity in meters/second,
n = propeller revolutions per second, and
D_P = propeller diameter in meters.

Note that for single, unducted propellers as used on most oceangoing ships, an effective propeller diameter of $0.707D_P$ should be used in Eqs. (4-28) and (4-29). The maximum flow velocity at a distance (x) behind the propeller, according to Blaauw and van de Kaa (1978), can be found from

$$U_{\max} = U_0 \left(\frac{2cx}{0.707D_P} \right)^{-b} \tag{4-28}$$

where b and c are coefficients that have average values of approximately 1.0 and 0.18, respectively, for meter and second units. Prop-wash currents in excess of 10 knots are possible at piles or bulkheads in ferry terminals. The drag force (F_c) on the pile can be found from the drag force Eq. (4-1), which reduces to

$$F_c = C_D U^2 A_p \tag{4-29}$$

where the current velocity is in feet per second, F_c is in pounds, and the pile projected area (A_p) is in square feet. C_D ranges from 0.6 to 1.2 for cylindrical piles to 2.0 or more for square and H-piles. For an otherwise heavily loaded or deteriorated pile, the added bending stresses could result in serious overstressing. PIANC (1987b) contains several valuable papers on prop-wash problems as related to bottom scour and harbor erosion, and PIANC (1997) provides guidelines for the design of armored slopes below wharves and useful discussion of propeller wash and bottom erosion in general. Hamill et al. (1999) present empirical equations and nondimensional graphs for predicting the maximum unconfined scour depths near quay walls caused by propeller wash, and Ryan and Hamill (2013) provide analysis of scour depths for end-on berthing, such as at Ro/Ro and ferry berths that include the effects of the vessels' rudder positions. Roubos (2007) addresses uncertainties in the design of bottom protection near quay walls, and general design guidance for protecting berthing structures from scour caused by ships is provided in PIANC (2015).

At sites with strong currents and/or exposure to wave action, scour may be an ongoing problem. A conservative estimate of scour depth for susceptible piles exposed to combined wave and current is 2 pile diameters, although 1 to 1.5 diameters is more likely. Wave and current action also create moving scour patterns in the seabed, such as sand waves and ripple marks. Scour dimensions and related problems, as well as potential solutions, are treated well in the *Coastal Engineering Manual* (USACE 2006), and Whitehouse (1998) addresses scour around marine structures in general. Sumer and Fredsoe (2002) provide textbook treatment of wave scour around marine structures.

Subsurface Conditions

The marine environment can present challenging seabed and subsurface conditions requiring in-depth geotechnical investigations. At certain river delta locations subject to the rapid accumulation of soft sediments, structures may be affected by mudslides and/or longer term soil flows. Kraft and Ploessel (1986) present a review of the stability of submarine slopes. Many waterfront structures are built on cohesive soils that are creep sensitive and subject to long-term settlements and displacements, as addressed by Gurinsky (2004). Another important potential subsurface hazard is soil liquefaction, which may occur under a variety of conditions, but especially earthquake or wave action. *Liquefaction* is the process of transforming any substance into a liquid state from an essentially solid state; it may occur around marine structures situated in submerged and even near-saturated granular material, such as seabed sands, primarily because of an increase in pore pressure and/or reduction of effective stress. Several types of liquefaction can be distinguished, depending upon the nature of loading, monotonic or cyclic, normal versus shear, readily drained versus undrained soil condition, and soil properties, including density and gas content. The physics of liquefaction around marine structures is described well by de Groot et al. (2006), and textbook treatment is provided by Sumer (2014). The ASCE/COPRI *Journal of Waterway, Port, Coastal, and Ocean Engineering* devoted two special issues (Sumer 2006, 2007) to liquefaction around marine structures; these issues include 15 papers based primarily upon a 3-year research program sponsored by the European Union (LIMAS 2004).

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CHAPTER 5

Berthing Loads and Fender System Design

The evaluation of vessel berthing energies and the design of protective fender systems are of central importance to the overall structural design of a facility. This area of design still leaves much to the designer's judgment, so it is most important that the basic mechanics and hydrodynamics of the berthing vessel and the fender and pier system response be well understood. The details of a fender's design can have a large effect on its need for maintenance and the ease with which required maintenance can be accomplished.

This chapter begins with an overview of the basic approach to calculating berthing energy and fender system design followed by more in-depth discussion of berthing parameters and alternative means of determining berthing energy. This discussion is followed by a review of current practice, the various types of fenders in use, and guidelines for the selection of specific fender types, sizes, and layouts. This chapter concludes with a discussion of fender materials and specifications.

5.1 Berthing Fundamentals

Berthing loads are imposed on a pier or mooring structure as a result of bringing a moving vessel to rest as it is maneuvered alongside into its at-rest berthed position. Bringing a moving object, such as a berthing vessel, to rest, as against a dock, requires that its kinetic energy be fully dissipated. This dissipation requires that work be done. Work is the product of a force acting through a distance (i.e., a force applied to a moving body). Some of the applied force that brings a berthing vessel to a stop may be provided by the vessel's own engines or berthing/maneuvering aids such as thrusters or tugboats.

The interface between the vessel and mooring structure is where marine fenders make their valuable contribution to a safe, low-maintenance marine facility. Marine fenders are energy-absorbing devices typically secured on the faces of marine structures for the purpose of reducing the forces inherent in arresting the motion of berthing vessels.

A properly designed marine fender is capable of deflecting far enough to dissipate all of the berthing energy without exerting a reaction force that will overstress the pier or cause permanent damage. Damage can take many forms and includes bending of steel, crushing or cracking of concrete, and crushing or breaking of timbers. All of these types of damage represent the standard "wear and tear" often in evidence at dock facilities with inadequate or nonexistent dock fenders. Dock fenders protect docks from berthing vessels but can also provide a high level of protection to vessels as well.

Since energy absorbed is mathematically defined as the area under a reaction versus deflection curve, there is an inescapable relationship among the three key parameters of marine fender design: energy to be absorbed, safe reaction limit, and deflection required to absorb the energy without exceeding the safe reaction limit.

Some important general considerations include vessel types; range of sizes and their geometry; facility type and operations, including frequency and manner of berthing; berthing approach conditions and environmental conditions affecting the maneuvering of the vessel; and fender type selection with regard to installation and maintenance, as well as first cost and longer term lifecycle costs. Fenders may also be subject to forces acting on the moored vessel because of environmental and operational loads and as such can be considered part of the mooring system as well. A properly designed fender system therefore enhances the safety, efficiency, and longevity of the facility.

Berthing Energy

The usual starting point in designing a dock fender system is assessing the kinetic energy of the berthing vessel. Given the large range in vessel sizes and the speeds at which they are capable of traveling, there is an almost infinite number of possibilities. Therefore, the designer's problem is to evaluate the most probable routine berthing maneuvers while providing an adequate margin of overload capacity to minimize damage under occasional high-energy accidental berthing. Factors affecting berthing energy to be absorbed by a fender include the following:

- The vessel's mass and velocity;
- The angle of approach;
- The relative water depth;
- Vessel controllability and tug assistance;
- Wind, current, and wave action;
- The structure's configuration; and
- The load/deflection properties of the structure and its fender system.

Even though, in theory, a small amount of energy is dissipated through the slight deflections of the vessel's hull and the actual dock structure, current fender-design practice assumes that almost all berthing energy is absorbed by the fender. An exception to this rule is in the case of fenders mounted on flexible structures, such as cantilevered monopiles that are designed to function as fenders and absorb energy by deflecting the pile or by the combination of the bending pile and a deflecting fender. The usual approach to fender design is to calculate the percentage of the vessel's kinetic energy that must be absorbed by the berthing structure and fender system and then evaluate the resulting force, based upon the load/deflection characteristics of the fender system for rigid piers or structure/fender system for flexible pier structures.

The work done by the deflecting dock/fender system is equal to the area under the load deflection curve, as given by the integral

$$E = \int_0^s R(s)\,ds \tag{5-1}$$

where
E = energy absorbed by the system,
R = reaction force, and
s = distance traveled.

Since the reaction of the fender determines the amount of pile deflection and energy absorption, selecting the optimum fender for such a combination may require several iterations of selecting a fender and then determining the deflection and energy absorption of the pier or pile for flexible structures. Many vintage timber piers were traditionally designed as flexible structures to absorb berthing impacts (Chellis 1961). Not only must the total energy be adequate, but also the pile(s) must not be stressed beyond a safe level.

In the design of contemporary fender systems, the deflection of the pier itself usually is neglected because the pier's movement is generally very small. Certain dolphins, or independent breasting structures, that have no fender system attached to them act themselves as fenders, and here the structure's deformation must expend all the energy. The designs of berthing dolphins and timber pile cluster dolphins are discussed in Section 7.5.

The energy to be absorbed by the fender system (E_F) is typically calculated using a deterministic approach by multiplying the vessel's total kinetic energy by a series of berthing coefficients, as given by

$$E_F = \frac{\Delta}{2g} V_N^2 \times C_e \times C_m \times C_c \times C_s \times C_g \tag{5-2}$$

where
Δ = vessel displacement;
V_N = component of vessel's velocity normal to pier;
g = acceleration of gravity;
C_e = eccentricity coefficient, which accounts for the vessel's rotation, depending upon the angle of approach and point of contact with the pier (it usually ranges from 0.5 to 0.8, and a value of 0.5 for quarter point contact is often is assumed);

C_m = Virtual mass coefficient, which accounts for added mass caused by entrained water (it usually ranges from 1.5 to 1.8, although it can be higher under certain conditions);

C_c = Configuration coefficient, which accounts for pier type and geometry (it usually ranges from 0.8 to 1.0);

C_s = Softness coefficient, which accounts for the relative stiffness of the vessel hull and fender system (it usually ranges from 0.9 to 1.0); and

C_g = Geometric coefficient, which accounts for the vessel's hull shape at point of impact and ranges from around 0.85 for convex curvature to 1.25 for concave curvature and is 1.0 for broadside contact. This coefficient is often ignored by many designers.

In the most extreme cases, equivalent to a vessel striking an open pier directly head-on or side-on, C_e and C_c would go to unity, and the total kinetic energy of the vessel plus entrained water would have to be expended in deforming the fenders, pier, and vessel's hull. For convenience herein, we can define an overall berthing coefficient C_b, which is equal to the product of the five coefficients in Eq. (5-2). The total energy (E_F) to be absorbed by the fender system is typically in the range of 60% to 90% of the vessel's total kinetic energy, or $C_b = 0.6$ to 0.9. Exceptions include the case of direct end-on berthing, where $C_b = 1.0$, and situations where the vessel under-keel clearance is very small and C_b may exceed 1.0. Fig. 5-1 illustrates the range of berthing energy versus vessel displacement for various normal velocities and a range of C_b. This figure can be used for preliminary estimates and to compare the effects of design velocity selection. The various berthing coefficients are discussed in further detail in the following sections. Note that the vessel displacement is the actual weight of the vessel during berthing. For a fully loaded vessel, this is the displacement tonnage (DT), as defined in Section 2.1. At certain sites with restricted water depths, larger vessels may be required to berth in partially loaded condition, in which case the displacement would need to be determined from the vessel's hydrostatic curves.

Selection of an appropriate normal velocity (V_N) is of utmost importance because the energy is proportional to the velocity squared, and this is one of the most subjective judgments a designer must make. In addition to being properly designed for the expected usual berthing impacts, fender systems must possess some degree of overload capacity to protect the ship and pier from damage under occasional accident-level impacts, generally referred to as "abnormal berthings," and to remain functional under a variety of wind, tide, sea, and operational conditions while the vessel is moored. Determination of V_N and abnormal berthing factors (C_{ab}) are discussed in further detail in Section 5.4. The fender system load/deflection properties may also be important in determining mooring loads under the influence of environmental forces while the vessel resides in its berth. Other factors to be considered before final fender selection are materials correction factors and manufacturing and performance tolerances, as discussed in Section 5.6. Guideline recommendations for the design of fender systems can be found in *Guidelines for Design of Fender Systems* (PIANC 2002), which has superseded the *Report of the*

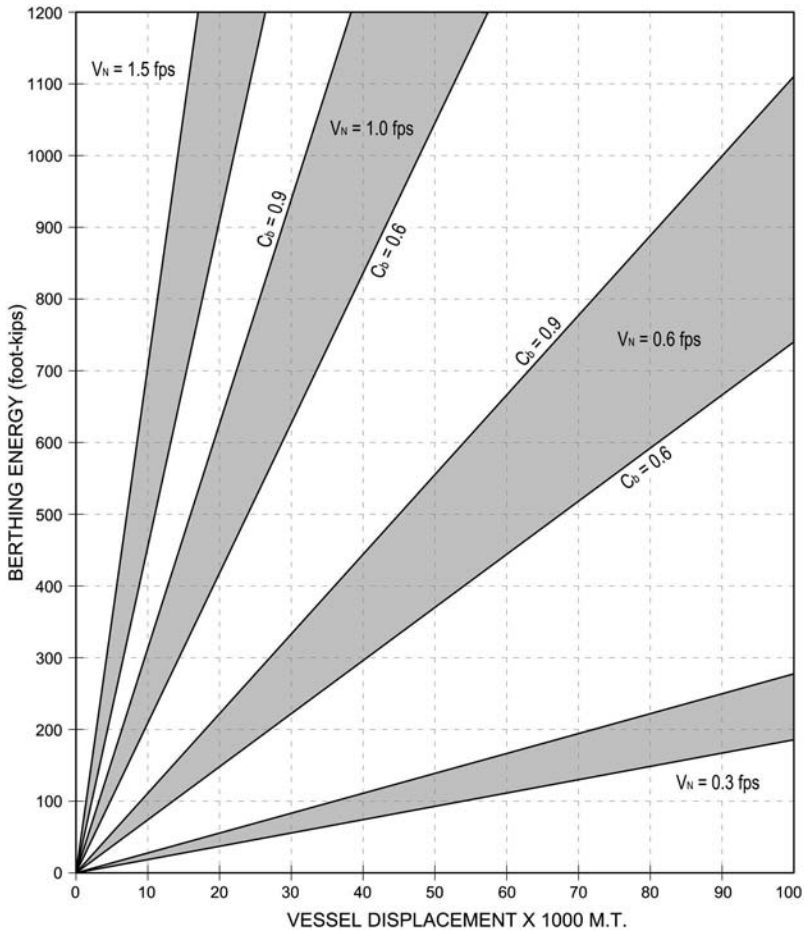

Fig. 5-1. Vessel berthing energy versus displacement curves

International Commission for Improving the Design of Fender Systems (PIANC 1984); however, the 1984 reference contains additional useful information not included in PIANC (2002). General guidance for the design of fender systems also can be found in DOD (2005), BSI (2013, 2014), EAU (2004), and ROM (1990).

The following section examines the various factors affecting the choice of design parameters, followed in turn by a discussion of their general application, as well as alternative methods of determining design berthing energy and fender system forces.

5.2 Berthing Ship Geometry

The general geometry of a berthing vessel is shown in Fig. 5-2. In most cases, the angle of approach (θ) between the vessel's longitudinal centerline and the pier face

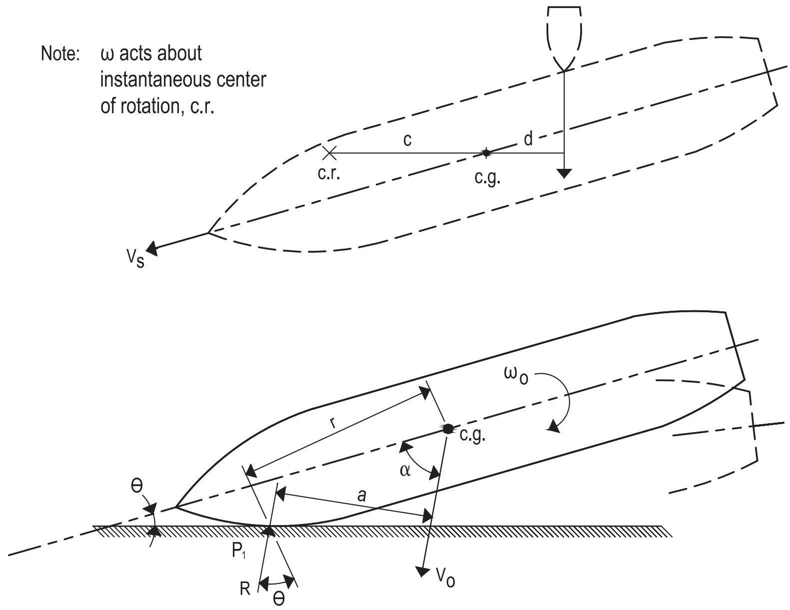

Fig. 5-2. Berthing vessel geometry

is taken as between 5° and 15° for design purposes, with the exception of certain ferry and roll-on/roll-off (Ro/Ro) berths and for large vessels, where θ is generally less than 3° to 5°. Note that for the case of a vessel proceeding ahead under its own power at velocity V, the normal velocity $V_N = V \sin\theta$. For large oceangoing ships, the vessel's final approach almost always is assisted or totally controlled by tugs and/or the ship's bow and stern thrusters, and the vessel's resultant sideways translational velocity, V_0, acts almost normal to the pier face. Under controlled conditions, the angle of approach is usually less than 3°. In addition to being translated sideways toward the pier, the vessel possesses some angular motion (ω_0) about a hypothetical "instantaneous" center of rotation (c.r.). The distance to the instantaneous center of rotation (C) from the vessel's center of gravity (c.g.) is related to the distance (d) from the c.g. to the applied force causing the rotation (e.g., tug thrust, wind, or current) by the following relation: $C = k^2/d$, where k is the radius of gyration of the vessel's mass about its c.g. For most vessels, k usually lies in the range of 0.20 to 0.29 times the vessel's length (L) and varies with the vessel's block coefficient (C_B), which is about 0.2 for $C_B = 0.5$, 0.25 for $C_B = 0.75$, and 0.29 for $C_B = 1.0$.

Referring again to Fig. 5-2, the angle between the vessel's resultant velocity V_0 and the vessel's centerline is denoted by α; the distance from a line drawn normal to the line of action of V_0 through the c.g. and the point of first impact P is denoted by α; and the distance from c.g. to the projection of P out to the vessel's centerline is denoted by r. It can be demonstrated (Vasco Costa 1989) that in relation to the above-defined geometric parameters, the amount of the vessel's kinetic energy to be absorbed by the fender system (neglecting the other berthing coefficients for the time being) is given by

$$E_F = \frac{\Delta}{2g} V_0^2 \left(1 - \frac{a^2}{k^2 + r^2}\right) + \frac{\Delta}{g} V_0 \omega_0 \left(\frac{k^2 a}{k^2 + r^2}\right) + \frac{\Delta}{2g} \omega_0^2 \left(\frac{k^2 r^2}{k^2 r^2}\right) \tag{5-3}$$

Neglecting rotation, Eq. (5-3) reduces to

$$E_F = \frac{\Delta}{2g} V_0^2 \left(1 - \frac{a^2}{k^2 + r^2}\right) \tag{5-4}$$

And, for the case of pure rotation,

$$E_F = \frac{\Delta}{2g} \omega_0^2 \left(\frac{k^2 r^2}{k^2 + r^2}\right) \tag{5-5}$$

Noting that the velocity of the point of the vessel's hull in contact with the fender $V_0 = \omega_0 r$ and substituting into Eq. (5-5) gives

$$E_F = \frac{\Delta}{2g} V_0^2 \left(1 - \frac{r^2}{k^2 + r^2}\right) \tag{5-6}$$

By comparing Eqs. (5-3) and (5-6), it can be noted that for equal velocities at the point of contact, the amount of energy (E_F) to be absorbed by the fender system is less for pure rotational motion than for translational motion. Therefore, in general, berthings made through rotating motion are to be preferred to those made through simple translation. It is also interesting to note that forces applied beyond the center of percussion (c.p.) of a rotating body do not change the reaction force at the point about which the body is pivoting. The distance dp from the c.g. to the c.p. is given by $dp = k^2/r$; therefore, tugs pushing on a vessel's stern beyond this distance from its c.g. do not increase the fender reaction. Eq. (5-6) also assumes that the angle of the reaction force (R) normal to the face of fender contact, denoted by ϕ, is larger than the angle of friction, such that sliding at the point of contact does not take place. Sliding expends additional energy and would help to reduce the reaction force. Depending upon the fender stiffness and V_0 and ω_0 at the instant of first impact, the vessel may bounce back and undergo a second impact nearer the vessel's stern. Second impacts not only are undesirable from a berthing operations point of view, but they may be of equal or greater magnitude than the first impact and thus must be considered in the evaluation of berthing impacts and in the layout and spacing of fender units. For ferries or Ro/Ros backing into a slip stern-on or head-on, there is little or no rotation, $\alpha = 0$, and all of the energy must be absorbed by the fenders. This is also the case for a vessel being translated normal to a pier and striking it perfectly broadside.

The discussion so far may seem somewhat academic from a design point of view, in that exact knowledge of all parameters is not possible. It is important, however, that the designer understand the mechanics of a berthing ship and be able to make reasonable assumptions about the above parameters under given circumstances. For a more rigorous analytical treatment of vessel berthing, refer to Vasco Costa (1964, 1989).

As a practical matter of design, the rotation of a vessel during berthing is accounted for by the eccentricity coefficient (C_e), already introduced, simplified to the following (Saurin 1963):

$$C_e = \frac{k^2}{k^2 + r^2} \tag{5-7}$$

In this formula, it is assumed that $\alpha = r$ for simplification of the analysis. The error associated with this assumption increases with an increase in the distance of the point of impact from the vessel's c.g. A more accurate formula, accounting for the effect of the angle α, is

$$C_e = \frac{k^2 + r^2\cos^2\alpha}{k^2 + r^2} \tag{5-8}$$

Often, for vessels berthing alongside, it is assumed that the point of first contact is at the quarter point, approximately 25% of the vessel's length, from the bow or stern, and C_e can be taken as 0.5. This distance almost coincides with the length of parallel midbody (see Section 5.6) for most large vessels. For vessels berthing against breasting dolphins, the point of first contact may be closer to around 35% length overall (LOA) from the bow or stern and C_e is about 0.7. For most vessels in loaded condition, the c.g. (or longitudinal center of gravity, l.c.g.) is located near the midpoint of their length between perpendiculars (LBP) or at their midships section. Vasco Costa (1989) notes that for large vessels in loaded condition, the c.g. is slightly forward of this point, and for ships in ballast it is usually aft of midships (see Section 2.2).

Fender reactions applied above or below the vessel's vertical center of gravity (v.c.g.) cause the vessel to heel, thus expending some additional energy. The heeling moment is given by the product Rh, where h is the vertical distance between the v.c.g. and the fender reaction (R). The resisting moment is given by $\Delta GM \sin\psi$, where GM is the transverse metacentric height (see Section 9.2) and ψ is the angle of heel (refer to Fig. 5-3). The energy expended in heeling the vessel can be shown in general to be small compared with the other factors discussed, and it is usual to neglect this effect in design.

The size and geometry of the structure to which the vessel is berthing also affect the energy, E_F. For closed piers, such as bulkheads and quay walls, water trapped between the wall and the approaching vessel that cannot escape quickly enough from around and under the vessel acts somewhat as a cushion between the wall and

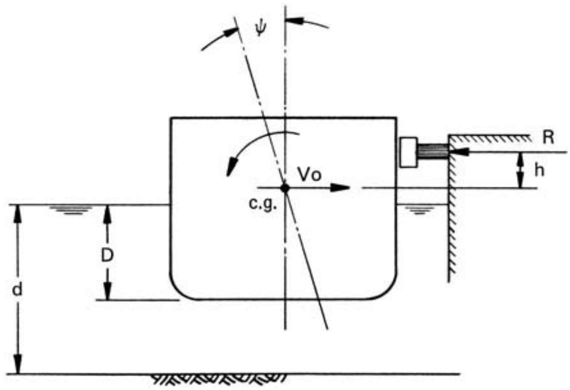

Fig. 5-3. Heeling geometry of berthing vessel

the vessel. This is especially true when the quay is much longer than the vessel and the water depth (d) to vessel draft (D) ratio (d/D) is small. Conversely, for open, pile-supported piers and especially for isolated berthing dolphins, the water moves freely past the pier, and no cushioning occurs. Piers and wharves may be considered semisolid for these cases: pile-supported wharves below which the bottom slopes rapidly upward, relieving platforms, piers shorter than the vessel's length, and so on. The following values for the so-called configuration coefficient (C_c) are recommended for design: For closed (solid) piers, $C_c = 0.8$; for semisolid piers, $C_c = 0.9$; for open piers, $C_c = 1.0$.

Values as low as 0.6 for closed construction longer than the vessel's length have been proposed (Kray 1982). Recent research efforts demonstrate that the cushioning effect of the pier configuration should be considered together with the added mass effect, as discussed in Section 5.3.

Other ways in which energy is dissipated upon impact include deformation of the vessel's hull and induction of vibrations in the vessel's hull and pier structure, which ultimately are released as heat. The latter factor is considered negligible, but it is often assumed that the vessel's hull absorbs about 5% to 10% of the energy of impact—in other words, the softness coefficient C_s ranges from 0.9 to 0.95. In certain instances, values of C_s from 0.5 to 1.0 have been used (Kray 1982), the lower value for cases of rigid fenders and flexible ships, and the value of unity for very soft fenders and rigid ships. Unless a more detailed study is conducted, a value of $C_s = 1.0$ is recommended where soft fendering is used, and a value of C_s between 0.9 and 1.0 where hard fendering is used, with soft and hard fenders defined as deflecting more than and less than 6 in., respectively, under the design impact energy (BSI 1994). The U.S. Navy (DOD 2005) applies an additional correction factor, the geometric coefficient (C_g), to correct for the ship's hull curvature at the point of impact. It varies from 0.85 for a convex curvature to 1.25 for a concave curvature near the vessel's ends and can be taken as 0.95 at the quarter point to 1.0 for broadside berthing along the ship's parallel midbody.

5.3 The Added Mass Factor

A body in an accelerated (decelerated) fluid flow is acted upon by inertial forces that resist changes in velocity. A vessel traveling at constant velocity is not opposed by inertial forces but by resistance caused by viscous effects and turbulent flow that create a pressure field around the vessel. When being slowed or suddenly stopped, the total force (F_R) resisting the change in velocity is given by

$$F_R = M_s \frac{\partial V_s}{\partial t} + \sum \frac{\partial (M_w V_w)}{\partial t} + R_v \tag{5-9}$$

where
M_s = mass of the ship,
M_w = mass of water moving with the ship,
V_s = velocity of the ship,
V_w = velocity of entrained water particles, and
R_v = resistance caused by velocity forces.

The mass of water supposed to move along with the ship is termed the *added mass* or *hydrodynamic mass*. The total mass of the vessel plus entrained water is termed the *virtual mass*. Likewise, a hydrodynamic mass coefficient (C_h) and virtual mass coefficient (C_m) can be defined, where the total virtual mass ($M_s + M_w$) is given by $C_m \times M_s$ or $(1 + C_h)M_s$.

The reader should be wary, when reviewing published data, about which form of the added mass is used. The virtual mass coefficient, C_m, is most commonly used and is used herein for convenience, allowing Eq. (5-9) to be written as

$$F_R = C_m M_s \frac{\partial V}{\partial t} + R_v \tag{5-10}$$

provided that we also assume that $V_s = V_w$.

Virtually all published values of C_m are based upon empirical and semiempirical methods. From classical hydrodynamic theory, the added mass of a cylinder oscillating in an accelerated flow is given by

$$M_w = \frac{\pi}{4} \rho D^2 \mathrm{LWL} \tag{5-11}$$

where
ρ = mass density of water ($= \gamma/g$, where γ is the unit weight of water),
D = vessel draft, and
LWL = vessel's length along the waterline.

This equation, which has been used for design purposes, particularly for ship-to-ship (STS) contacts, usually gives corresponding values of C_m on the order of 1.5 to 1.6 for typical larger vessels such as tankers.

One of the most commonly used formulas for C_m, as recommended by Vasco Costa (1964) after the work of Grim (1955), is

$$C_m = \left(1 + \frac{2D}{B}\right) \tag{5-12}$$

where B is the vessel's beam. This empirical equation usually yields values of Cm on the order of 1.3 to 1.8 for most oceangoing vessels and was adopted by BSI (1994) for conditions where under-keel clearance (UKC) >0.1 and $V_N > 0.08$ m/s (0.26 ft/s). Another useful formula for design applications is that of Ueda (1981), which is a modification of Eq. (5-11) and includes the vessel's block coefficient (C_B), which results in higher C_m for finer (lower C_B) vessels:

$$C_m = 1 + \pi D/2C_B B \tag{5-13}$$

None of the foregoing formulas, however, considers the importance of relative water depth as given by the ratio d/D and other potentially important parameters such as the vessel's initial velocity, the proximity of solid structures, the deceleration distance, and fender system stiffness. PIANC (2002) compares typical results from Eqs. (5-12) and (5-13) and recommends the following in lieu of more specific information:

- For very large under-keel clearances where UKC > $0.5T$, take $C_m = 1.5$;
- For small UKC < $0.1T$, take $C_m = 1.8$;
- For intermediate values of UKC, use linear interpolation; and
- For end-on berthing, take $C_m = 1.1$.

In open water without defined boundary conditions, the added mass of a vessel in forward motion is on the order of 10%, or $C_m = 1.10$ (Kray 1982), as recommended by PIANC (2002). According to AASHTO (2010) bridge collision criteria, C_m for head-on collisions ranges from 1.05 for UKC/$D \geq 0.5$ to 1.25 for UKC/$D \leq 0.1$, with linear interpolation for values in between.

Various investigators have determined extreme values of C_m ranging from around unity to near 16.0. The higher values generally have been based on model experiments and have not been reconciled with practical experience. Qualitatively, however, they are very instructive in terms of demonstrating the effects of under-keel clearance and other boundary conditions.

Notable investigations of added mass effects that are relevant to vessel berthing impacts include Giraudet (1966), Lee et al. (1975), Hayashi and Shirai (1977), Blok and Dekker (1979), Ueda (1981), Ball and Hall (1982), and Ball and Markham (1982). Mathematical models also have been developed to treat the berthing impact problem analytically, such as those presented by van Oortmerssen (1974) and Fontijn (1980). Middendorp (1981) developed an analytical solution using long-wave theory to describe the movement of water around the berthing ship.

Although convenient for some purposes, the concept of added mass as entrained water lumped together with the vessel's mass is somewhat misleading. A lucid but simplified explanation was given by Ball (1982). For a vessel moving sideways (sway motion), the rate of displacement of water forward is balanced primarily by a return flow under its keel and around its ends. When the water depth is shallow relative to the vessel's draft, more water is forced to flow around the vessel's

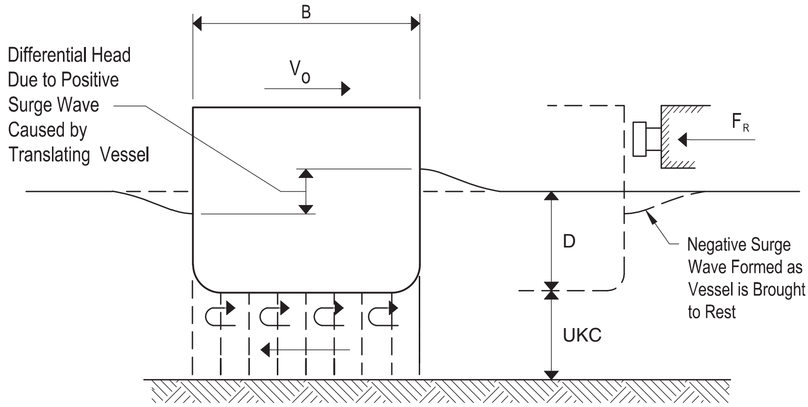

Fig. 5-4. Added-mass definition sketch

ends. When the vessel is brought to rest more or less suddenly, the displacement flow is stopped, but the momentum of the moving water (i.e., the under-keel flow) continues, creating, in effect, a negative pressure wave inboard and a consequent drop in water level and a positive pressure wave and rise in water level on the vessel's outboard side (Fig. 5-4). Ball refers to these as surge waves, which result in a difference of water levels across the ship, forcing the vessel against the fender. Because the surge force is time dependent, the peak surge lags the instant of impact, so that the total added mass felt by the fenders depends upon their stiffness and rate of load application, the hydrodynamic boundary conditions, and, importantly, whether or not the vessel was accelerated or decelerated immediately before striking the fender. A seeming paradox is that for a given impact velocity, a ship produces more fender energy (E_F) if slowed down just before impact than if accelerated beforehand. Reduced under-keel clearance results in an increased under-keel flow rate, and it has the effect of increasing the time over which the surge force acts, thus increasing E_F.

Ball and Markham (1982) reported on scale-model tests undertaken to evaluate the relative importance of various parameters and emphasized the need for full-scale verification. Some of their conclusions, based upon a 1/142 scale model 100,000 dead weight (DWT) tanker driven sideways onto a fender of average prototype stiffness and at a prototype velocity of 0.1 m/s, are the following:

1. Under-keel clearance in the range of 2.5% to 5% of draft ($d/D = 1.025$ to 1.05) results in C_m values much higher than those that are usually applied.
2. The presence of a solid wall within distance B of the fender face may significantly increase C_m; a separation of approximately $B/4$ has the greatest effect.
3. The vessel does not have to be moved great distances to develop large C_m values; approach distances of approximately $B/2$ have the greatest effect.

The second item is contrary to the usual practice of reducing E_F by the configuration coefficient for solid structures. Another seeming contradiction is that soft fenders with large deflections actually increase the added mass for a given velocity of impact. Clearly, there is a need for more definitive research and full-scale corroboration of results in this area of study. As is seen in Chapter 6, added-mass effects are important to the determination of mooring loads, and a vessel has different added mass relationships for movements or rotations about its various coordinate axes. This discussion has been limited to added mass in sway, although yawing, or rotation about a vertical axis through the vessel's c.g., also is important to the berthing ship.

Hayashi and Shirai (1977) recognized the importance of velocity flow effects in their studies of tanker models and also found high values of C_m for low d/D ratios. They gave their results in terms of the nondimensional Froude number $N_F = \frac{V_N}{\sqrt{gd}}$ (see Section 6.10). Their results were presented for N_F ranging from 0.01 to 0.07 and d/D from 1.0 to 2.1. Beyond $d/D = 2.0$, this parameter appears to become insignificant. Hydrodynamic mass is inversely proportional to N_F and V_N; therefore, with a decrease in V_N, the N_F decreases, and, consequently, C_m increases for any depth of water.

The evaluation of a vessel's virtual mass under given conditions is an area for further study; field verification is most important. As for current design practice, it appears that the extreme values associated with $d/D \leq 1.1$ are unlikely to be attained. The configuration coefficient should perhaps be taken into account in the calculation of C_m and not applied separately. The designer is left with a choice of applying conventional methods of calculation such as Eqs. (5-12) or (5-13) or the simplified PIANC criteria, or reviewing the literature to find data most applicable to the particular problem at hand. Where under-keel clearances are small ($d/D \leq 1.1$) and large vessels are being considered, special consideration should be given to a more rigorous evaluation of C_m. For catamaran hulls, the added mass may be significantly higher than for monohull vessels, considering the generally higher beam-to-length ratio and the effect of water trapped between the hulls. Model tests or numerical modeling of fluid flow may be warranted. PIANC (2002) presents a modification to the traditional formulas used to calculate the added mass for preliminary estimates. Additional criteria for determining added mass of U.S Navy vessels can be found in NFESC (n.d.). The PIANC (1984, 2002) reports on fender systems review other studies on this subject and provide additional recommendations for design.

5.4 Berthing Impact Energy Determination

Berthing impact energy levels to be used for fender system selection and design can be determined by one or more of four basic approaches to the problem. In the conventional, deterministic calculation procedure, the kinetic energy of the moving vessel is multiplied by various coefficients that account for specific assumed design conditions. This approach is most commonly used and has already been introduced, and the subsequent discussion of the various coefficients serves to illustrate the many factors involved. An alternative approach is to calculate the force-deceleration time history against the fender system elastic deformation to determine peak reaction forces directly. Where sufficient field measurements exist, a statistical approach may be taken, making use of the fact that the frequency distribution of impact energies measured at various berths approximately follows a log-normal distribution. Such analysis generally is valid only for a specific harbor and vessel size and type. The empirical approach involves the application of simple formulas based upon past experience and certain assumptions relating the vessel's displacement or other characteristics directly to the berthing energy. Mathematical models have been developed that attempt to model the entire process and allow analysis of a wide range of input conditions. These models need to be calibrated through actual measurements under controlled conditions or via physical model experiments. The direct use of physical models to determine berthing energies cannot be applied with full confidence under the current state of the art. This section reviews the application of these four methods.

Conventional Calculations and Berthing Velocities

The conventional method of berthing energy calculation has been presented as Eq. (5-2), and various factors relating to the use of this equation have been explored. The remaining and single most important factor in its application is the selection of an appropriate velocity of approach (V_N). The velocity at impact is affected largely by vessel controllability and the skill and techniques of those in charge. The number, type, and use of tugs, the use of bow thrusters, weather conditions (including visibility, wind, sea state, and the presence of currents), the maneuvering space available, and the frequency of berthing affect design velocity selection. Docking aid systems (DASs) are used at some large vessel facilities, especially at exposed locations, to monitor the vessel's speed and distance off. Docking aid systems typically have large display panels mounted on the berth structure (Fig. 5-5) and may also transmit data directly to the vessel as well as to a shoreside monitoring station. Measurements are typically made by pier-mounted laser devices using differential GPS (DGPS) or real-time kinematic GPS (RTK). Portable DAS devices are also available. Measurements from more than one location can be used to determine the vessel's angle of approach and rate of rotation, as well as normal approach velocity. Berthing aid systems should be required at all facilities that berth vessels of 200,000 DWT and over (BSI 2000). A general discussion of their operation and advantages is presented by Tomlinson (1986) and PIANC (2012).

Normal velocities used in design generally are within the range of 0.25 to 1.5 ft/s (8 to 46 cm/s); even higher speeds are associated with smaller vessels. Various criteria have been presented for the selection of design velocities with regard to exposure, approach conditions, vessel displacement, and other factors, as described

Fig. 5-5. Docking aid system (DAS) display board mounted on mooring dolphin showing vessel distance off and approach speed at fore and aft locations. Note red, yellow, and green warning lights

Source: Photo courtesy of Trelleborg Marine Systems

in the following discussion. Brolsma et al. (1977), based upon the earlier work of Baker (1953), demonstrated the use of probability laws in the determination of design velocities and produced curves of V_N versus vessel displacement for various berthing conditions related to the difficulty of berthing based on exposure and navigation conditions (Fig. 5-6). These curves have been adopted by BSI (1994) and EAU (2004) and are in popular use. A discussion of the development and application of Brolsmas' curves is provided by Beckett Rankine (2010). PIANC (2002) provides design guidance on selection of V_N and presents a table of recommended mean values, taken to represent the 50% confidence level, taken from the Spanish ROM (1990) to be used in absence of better data. Values range from 16 to 60 cm/s in favorable to unfavorable conditions for vessels up to 10,000 DT and from 8 to 20 cm/s in favorable to unfavorable condition for vessels over 50,000 DT. These values assume that tug assistance is available and can be higher without tug assistance. It can be inferred from the tables in the PIANC and ROM references that 8 cm/s is a minimum that should be considered. The California Building Code (MOTEMS 2011) also provides minimum design values for V_N based upon berthing risk conditions and vessel displacement with a minimum under favorable conditions of 8 cm/s for vessels >100,000 DWT and up to 40 cm/s for vessels <10,000 DWT under unfavorable conditions.

Fig. 5-7 illustrates V_N versus vessel displacement for varying exposure conditions based on U.S. Navy practice (DOD 2005) and is often used in North American design

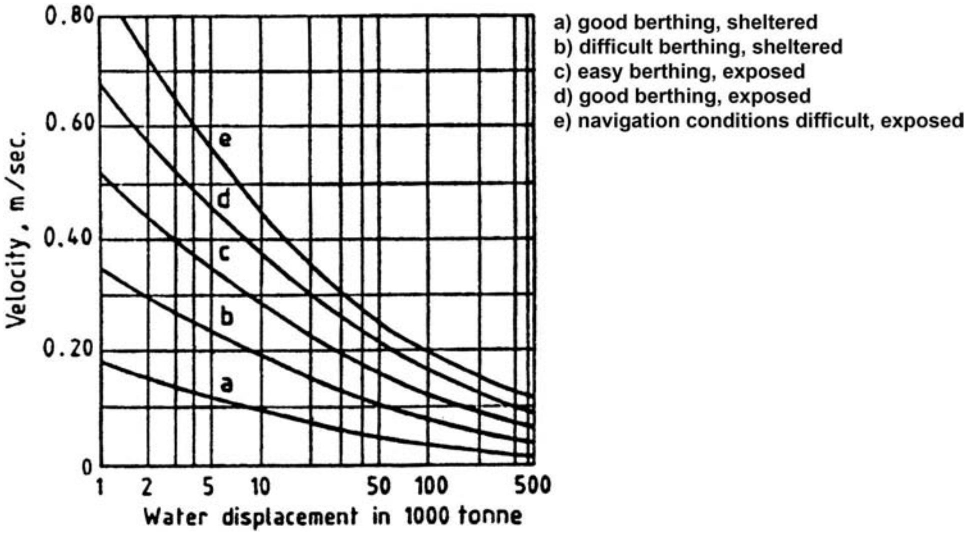

Fig. 5-6. Berthing velocities for fenders system design versus vessel displacement for varying navigation conditions

Source: BSI (1994); reproduced with permission from BSI Standards Ltd.

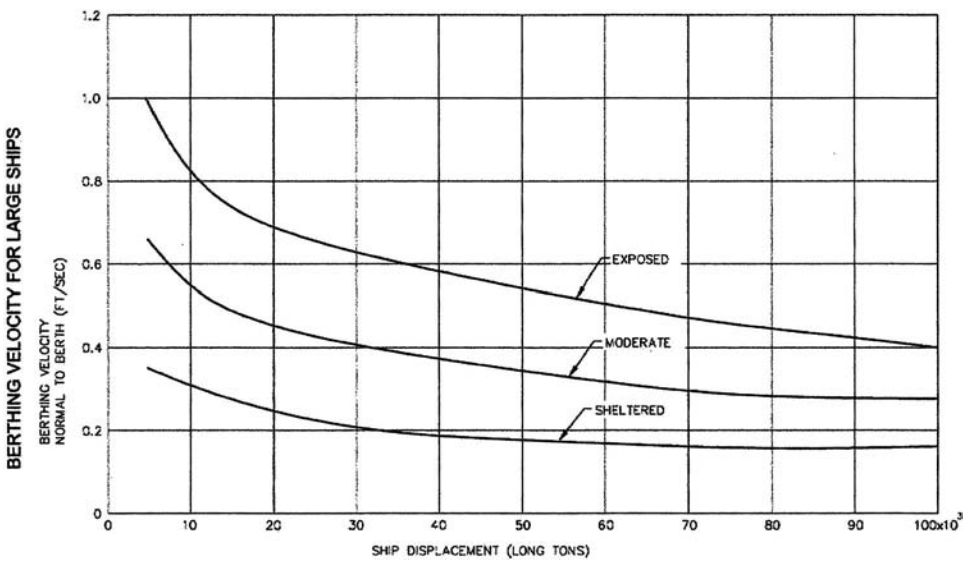

Fig. 5-7. Vessel normal velocity versus displacement

Source: DOD (2005)

practice for commercial facilities as well. A mean value of V_N from 0.3 to 0.5 ft/s is representative for the larger vessels. Table 5-1 summarizes some actual measurements taken at several berths in Japan. More recent field measurements by Hein (2014) conducted at the port of Bremerhaven, Germany, on large containerships found a mean value of $V_N = 5$ cm/s with a peak value up to 17 cm/s based

Table 5-1. Berthing Speed

Operating Conditions	Berths	Ship Size (DWT)	Berthing	Berthing Speed (cm/s) Range	Average	n
	T company (K berth)	75,000–250,000	First impact	1–13	5.3	50
	T company (S berth)	50,000–245,000	First impact	1–10	4.3	59
Case 1	N company (No. 3)	21,000–280,000	First impact	1–8	3.3	36
	N company (No. 4)	160,000–480,000	First impact	1–5	2.3	12
	N company	60,000–170,000	First impact	2–9	2.3	16
Case 2	K company	155,000–250,000	First impact	1–10	4.3	8
	O company	100,000–250,000	First impact	1–7	3.7	30
Case 3	I company	100,000–220,000	First impact	2–105–28	4.517.1	1918

Notes: Case 1: Berth located in harbor at which operating conditions are moderate.
Case 2: Berth facing open sea at which operating conditions are relatively severe.
Case 3: Open-sea berth at which operating conditions are severest. 1 cm/s = 0.0328 ft/s; n=number of data.
Source: Bridgestone Tire Company (n.d.); reproduced with permission from Bridgestone.

upon radar measurements of more than 1,000 berthing maneuvers. These are significantly lower than recommended design velocities; 98% of berthings are 25% lower. Yamse et al. (2014) report similar results with respect to V_N for containerships at Japanese and East South Asia ports. Yamse et al. further evaluated the effects of vessel DWT, tug and thruster assistance, and wind and wave action at one bulk handling terminal. They noted little effect on V_N caused by wind and wave action that was within the specified facility operating limits as well as little correlation with DWT and tug and thruster assistance.

Design velocities for ferry and Ro/Ro operations that feature high-frequency berthings are generally much higher than these values. Such vessels usually are guided into their slips by outer guide-in dolphins or fender racks and then by wharfside fenders before finally contacting the head dolphins or end fender units end-on. Recommended design velocities for outer guide-in dolphins range from 2.0 to 3.0 m/s (PIANC 2002), or approximately 6 to 10 ft/s, to 0.5 to 1.0 m/s (1.6 to 3.3 ft/s) for wharfside fender units where the vessel makes a direct approach. These velocities usually are associated with a minimum approach angle of 10° to 15°. The actual velocity used should consider the vessel's stopping distance in relation to the slip length. Lower velocities apply when the vessel is essentially stopped and worked into the slip or alongside in a transverse mode. The end fenders should be designed for approach velocities of from 0.5 to 1.0 m/s (1.6 to 3.3 ft/s) when a direct, end-on approach is made. It is customary to select a design velocity representative of usual conditions with some overload capacity for high-energy berthings and anything beyond, which is considered an accident. BSI (2000) notes that the extreme velocities from record data are on the order of four times the most favorable berthing values. In regard to accidents, economic analysis can be made of various fendering systems for different levels of energy if reasonable assumptions can be made about the probability of damaging accidents over the structure's life (Vasco Costa 1964, Wirzbitzki et al. 1977).

The angle of approach (θ) is related to the normal velocity by the following: $V_N = V_s \sin\theta$, where V_s = the forward speed of the vessel and is typically less than about 5° for larger vessels being maneuvered by tugs, whereas angles of 10° to 15° generally apply to smaller vessels making an approach under their own power. The angle of approach is also important in determining the eccentricity coefficient and the reduction of energy absorption of a fender. Hein (2014) and Yamse et al. (2014) found that approach angles for large containerships were typically less than 1°, and Hein further notes that berthing impacts engaged multiple fender units. Angles as high as 20° have been proposed for ferry slip design (DOD 1988). Jahren and Jones (1996) report on design criteria proposed for the Washington State ferry system focused on end-on berthings, including review of design velocities and berthing coefficients. Jahren and Ishii (1995) reported on conceptual alternative designs for accidental and emergency berthings at ferry terminals. The PIANC Working Group, WG-145, "Berthing Velocities for Fender Design," is actively collecting data and reviewing berthing practices, and publication of updated information and design guidance can be expected in the future. Burkhart and Matakis (2013) present some early results for tankers, bulk carriers, and containerships conducted at two U.S. major ports using a portable DAS system.

Given the many uncertainties and assumptions made in the conventional approach to determining V_N, it is often advisable as recommended by PIANC (2002) to apply an abnormal impact factor (C_{ab}) to the calculated design berthing energy before fender selection. Abnormal berthing impacts can result from human error, equipment failure, and/or adverse environmental conditions, and these effects must be evaluated with regard to their effect on the fender system and berth structure. The vulnerability of the fender system and support structure and the consequences of fender failure, berth downtime, hazardous cargoes, and frequency and variety of vessels using the berth must all be considered. Selection of an appropriate value for C_{ab} is subject to the level of confidence in the energy calculations and is usually within the range of 1.1 to 2.0. PIANC (2002) general guidance is summarized as follows:

- Tankers and bulk carriers: 1.25 largest to 1.75 smallest
- Containerships: 1.5 largest to 2.0 smallest
- General cargo: 1.75
- Ro/Ro and ferries: ≥2.0
- Tugs and workboats: 2.0

Further discussion of the abnormal impact factor can be found in PIANC (2002). The U.S. Navy (DOD 2005) recommends increasing the calculated berthing energy by at least 50% to account for abnormal impacts. An additional consideration not discussed in the PIANC report is that by applying too high a value of C_{ab} and thus conservatively selecting a fender that is too stiff, the structure may be realizing repeatedly higher reaction forces under normal berthing impacts than necessary. Also worth considering is that with less cushioning effect, passengers on ferries for example may be subject to unnecessary jostling on a more frequent basis than if a softer fender were selected. The designer should therefore use discretion in applying abnormal impact, overload factors to avoid a potentially overdesigned fender system.

Force-Deceleration Method

An alternative approach to the conventional calculation method is the force-deceleration method, whereby the time history of the vessel's deceleration is equated directly to the stiffness and elastic deflection of the fender system and/or berthing structure. Derucher (1983) applies this approach to the general evaluation of pier fendering systems, and Sherman (1980) demonstrates the approach in the design of a ferry fender rack. This approach is most suited to computer applications, as described in the following discussion on mathematical modeling.

Statistical Approach

The conventional method of energy calculation leaves many difficult decisions up to the designer, especially for berths frequented by vessels of different sizes. It should be intuitively obvious that, whatever the level of energy selected for design, there is always some risk that the energy level will be exceeded during the structure's life (Eddie 1981). The decision then as to what E_F to design for must consider the usual range of E_F under normal berthings, the overload capacity required to accommodate some infrequent level of high-energy berthings, and the probability and risk of damage caused by accidents associated with low probability levels. These constraints strongly imply the application of statistical techniques, but the main problem with this approach is with the availability of a sufficient number of actual measurements relevant to a given berthing situation. Actual measurements of berthing energies are reported by Goulston (1973) and Svendsen (1970); the latter finds a direct correlation with ship size and current conditions as the most significant parameters.

Dent and Saurin (1969) and Toppler and Weersma (1972) propose a statistical approach based upon many direct measurements. Dent and Saurin recommend the following energy levels for design of a moderately exposed tanker berth:

- An E_F of 16.8 ft-kips per 1,000 DWT of design vessel for fenders at the yield stress;
- An E_F of 11.2 ft-kips per 1,000 DWT of design vessel under normal maximum conditions, corresponding to the working stress in fenders and berthing structures; and
- An E_F that is 5/8 of the above for protected berths and also for loading terminals where vessels normally arrive in ballast condition.

Toppler and Weersma (1972) note that the following conditions must be accepted in developing a rational statistical approach: berthing energies in excess of the ultimate design energy may occur, and their occurrence is related to the number of berthings during the lifetime of the structure; the size distribution of vessels, in particular the maximum and average size; and their relative frequencies, as well as the site conditions. They further note that once a sufficient volume of statistical data is available, a risk factor can be determined that can assign probabilities of damaging impact, which can be weighed against structure first cost and lifecycle cost. They then propose the following design energy levels for tankers greater than 100,000 DWT:

- An E_F of 2,000 ft-kips per 100,000 DWT for low-frequency berthing (i.e., less than 40% of all berthings),
- An E_F that is 1.25 times the above for high-frequency berthing (i.e., greater than 60% of all berthings),
- An E_F equal to 85% of the above for loading terminals (vessels in ballast) and sheltered locations, and
- An E_F equal to 110% of the above for very exposed locations.

Fig. 5-8 summarizes their proposed design criteria.

The British Petroleum Company has taken an active role in conducting actual measurements at its marine terminals, as reported by Balfour et al. (1980). Balfour and his colleagues grouped vessels into various size ranges and berth types and then produced log-normal plots of energy versus probability of exceedence, which plot as almost straight lines. They applied a correction factor (C_f) for fender spacing, acknowledging the importance of the distance from the center of mass to the point of fender contact. The correction factor is given by

$$C_f = \frac{C_e \text{ for standard } 0.15\,LOA \text{ eccentricity}}{C_e \text{ for eccentricity of half fenders pacing}}$$

Balfour and colleagues further note, however, that extreme impacts are not necessarily associated with center-of-mass berthings. For design purposes, they suggest that the maximum impact to be resisted should be that corresponding to a return period equal to the structure design life. Their base case is assumed as a 30-year structure life with 100 berthings per year, for a total of 3,000 berthings. The associated probability of occurrence, then, is 0.033% or 1/3,000. Fig. 5-9 illustrates the application of their method to two particular piers in Rotterdam Harbor. Fig. 5-10 is a generalized recommendation for design energies versus displacement for any enclosed harbor. Balfour et al. also found that the berthing energy is related to the inverse square of the vessel displacement. Ueda et al. (2001) present a more recent discussion of statistical design of fenders.

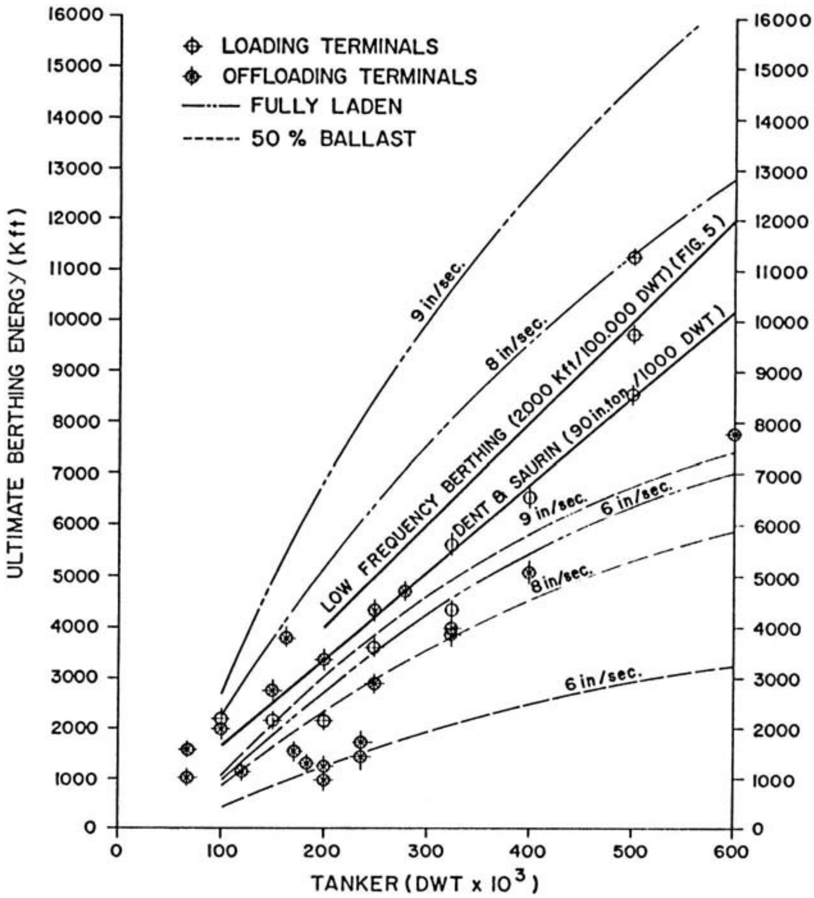

Fig. 5-8. Ultimate berthing energies for tankers

Source: Toppler and Weersma (1972); reproduced with permission from Offshore Technology Conference

Empirical Methods

The empirical approach is best suited to specific sites where the relative berthing frequency, vessel characteristics, berthing conditions, and so on remain fairly constant. Empirical relationships or formulas can be developed from record data and past experience. Girgrah (1977) proposes the following empirical relation based solely on vessel displacement:

$$E_F = \frac{\Delta}{120 + \sqrt{\Delta}} \text{ for } E_F \text{ in metric ton-meters} \tag{5-14}$$

This equation applies to fenders located at the point of first contact and can be reduced 50% for interior fenders. Piaseckyj (1977) notes that the relationship can be adapted to suit a given designer's experience by varying the constant, here given as 120.

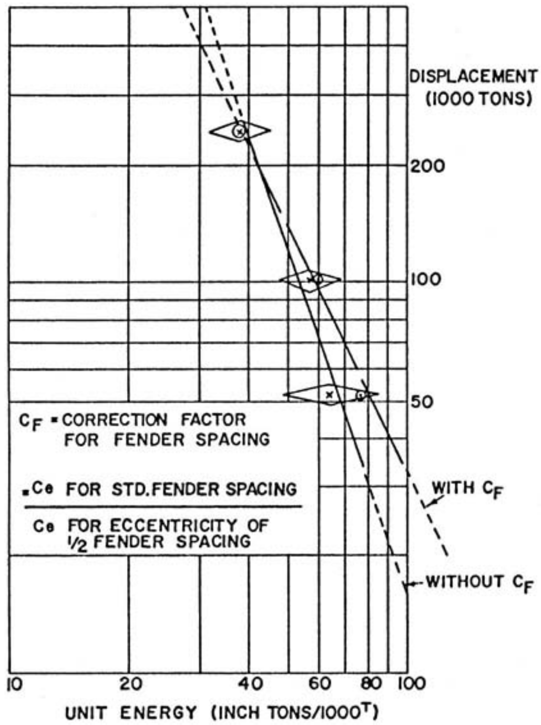

Fig. 5-9. Berthing energy versus displacement for 1:3,000 probability of occurrence

Source: Balfour et al. (1980); copyright ASCE

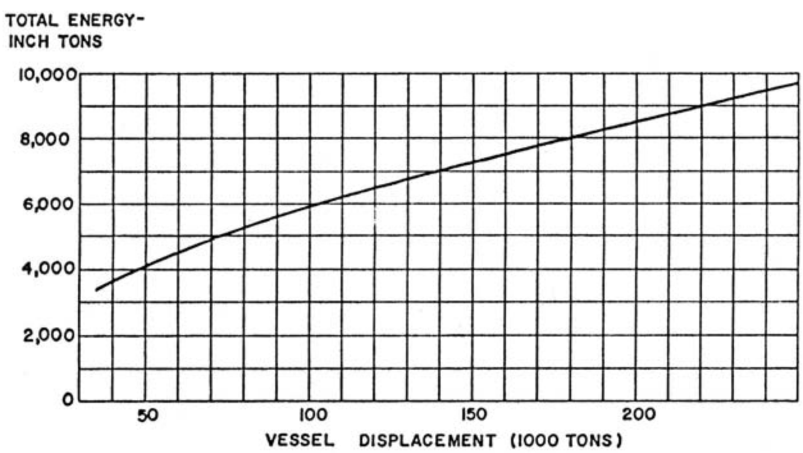

Fig. 5-10. Design energy values for an enclosed harbor

Source: Balfour et al. (1980); copyright ASCE

Mathematical Models

Mathematical models have been developed for predicting berthing and mooring forces and are discussed further in Section 6.9. PIANC (2002) includes a general description of computer models applicable to the berthing ship. Notable efforts in

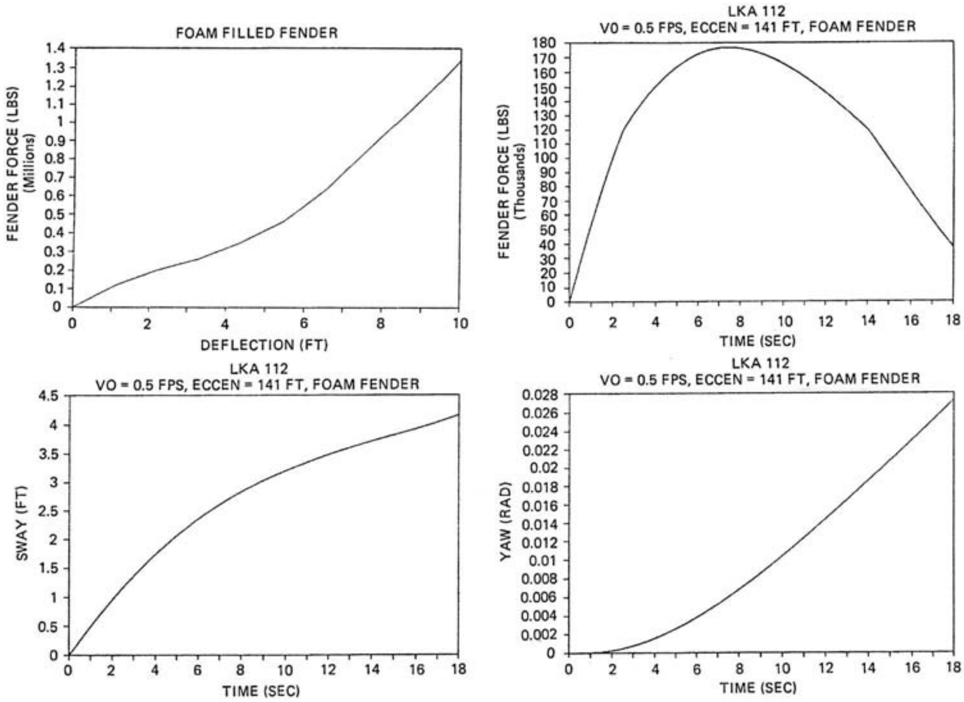

Fig. 5-11. Load-deflection curve and time histories of fender force, vessel sway, and yaw for a foam-filled fender based on numerical berthing model of Fontijin (1980)

Source: Courtesy of John R. Headland

this field include Lee et al. (1975), van Oortmerssen (1974), Fontijn (1980), and Middendorp (1981). Bradshaw et al. (2006) developed a simple dynamic model for fender pile analysis. Further description of the analytical treatment of berthing vessels can be found in various papers of the NATO Advanced Studies Institute proceedings (NATO 1965, 1973, 1987).

An example output from a numerical berthing model based on Fontijn's work (1980) is given in Fig. 5-11 for a 17,500-DT naval vessel berthing against a foam-filled fender unit. The initial approach velocity of the vessel was 0.5 ft/s, and the impact occurred at a point one-fourth of the length of the ship away from the vessel's center of gravity. The figure presents a load-deflection curve for the foam-filled fender, as well as time histories of the fender force, vessel sway, and vessel yaw during the berthing impact. It is important to note that because fender systems may have a major influence on mooring loads, their design should integrate both berthing and mooring conditions. Mathematical models could provide a detailed description of the interrelationship of berthing and mooring problems, but the practical problems of physically modeling the impact process under a wide range of conditions in order to calibrate the models have perhaps inhibited development.

In summary, several methods are available for the determination of berthing impact energy and/or the resultant forces as they relate to marine fender system design. The conventional calculation method remains the most generally applicable approach that a designer can readily implement. Where a sufficient database exists, and/or where there is a long record of experience such as within developed harbors with similar facilities, then the statistical or empirical methods offer important advantages. At remote and exposed sites for large vessels and/or where major facilities are planned, mathematical models offer the advantages of considering a wide variety of conditions and integrating berthing and mooring design requirements.

5.5 Fender System Types and Selection

A wide variety of fender system types are available to the designer. In addition to commercially manufactured units, many other options are open to the designer developing custom-designed fender systems, depending upon specific facility requirements. Fender systems may dissipate impact energy in various ways: through conversion of kinetic energy to potential energy, as in gravity and buoyancy-type fenders; by elastic deformation in compression, bending, shear, and/or torsion, as in elastomeric-type units; by conversion of energy into heat by friction, as in hydraulic units; and even by the plastic deformation of certain expendable-type fenders. Various fender system types and their characteristics are described in detail in DOD (2005), PIANC (2002), and BSI (1994, 2014), and some of the more common types are described later in this section.

The selection of the optimum fender type for a given application depends upon the following factors:

- Energy absorption requirements;
- Maximum allowable reaction force;
- Vessel standoff distance requirement;
- Effect of angular impact on performance;
- Allowable hull pressure on the vessel;
- Coefficient of friction and vertical and longitudinal rubbing forces;
- Range of vessel sizes and hull shapes;
- Range of tide and exposure conditions;
- Environmental exposure effects;
- Frequency of berthing and wear considerations;
- Factor of safety and overload capacity;
- Cost and long-term maintenance/repair costs; and
- Local availability, costs, and construction practices.

For a given berthing energy (E_F), the terminal force or reaction (R) on the pier structure and vessel hull is a function of the load/deflection characteristics of the pier and fender system. In most cases, except for certain dolphins that absorb the impact by their own deflection, the deflection and hence the energy absorption by the structure itself are neglected. Fenders usually are designed for some prescribed level of berthing impact energy, and, in general, it is desirable to minimize the reaction-to-energy ratio (R/E) of the fender system. The lower the R/E, the more "efficient" is the fender. When hull pressure is an issue, the use of lower R/E fenders often also reduces cost. Current practice recommends specific ranges of nominal hull pressures for various types of vessels. These limitations are then met by dimensioning the contact surfaces of fenders so that their rated reactions divided by their nominal contact surface areas do not exceed their specified limits. It should be understood that these limitations are rules of thumb rather than actual pressures because many variables go into determining the actual contact pressure at any given point. In reality, many times the theoretical contact pressures are infinite because the vessels make only line contact with the fenders. Line contact situations occur when vessels make angle contact near the bow or stern. This is, in fact, the most common mode of berthing. When a curved part of a hull makes contact with a rigid contact panel of a fender system, there can be no more than line contact down the vertical center of the panel. When a vessel with hull belting makes contact with a fender panel in such a manner that a belt contacts a panel near either its top or bottom extreme, the opposite end of the panel makes line contact with the hull. Thus, the important thing is not the absolute value of the actual local contact pressure, but to provide a contact surface of the recommended size. It can be assumed that when rigid contact panels breast against vessels, the loads are most often actually transmitted primarily to hull framing, not to hull plating. This assumption works because rigid contact panels bridge between areas of greater support. Because they are more rigid than hull plating and because they span relatively large areas, it is actually very difficult to dent hulls with rigid panels. In fact, most denting comes from fenders that either have small contact areas, and thus can potentially concentrate contact loads in the middle of spans between structural supports, or from fenders with contact surfaces that are more flexible than hull plating. These also are capable of exerting high loads between structural supports.

It should be noted that fenders play an important role in the loads imposed upon moored vessels, and their characteristics must be checked against mooring requirements, as discussed in Chapter 6 and in Section 5.6.

Fender systems can be classified as to their general load-deflection characteristics, as shown in the generalized curves of Fig. 5-12. The area under the curve is equal to the work done in deforming the fender, and hence to E_F. Eq. (5-1) can be rewritten to express the relationship between R and E_F as

$$E_F = \int_0^{\delta} R(x)\,dx \tag{5-15}$$

where
x = deflection, and
δ = deflection at the rated or desired energy level.

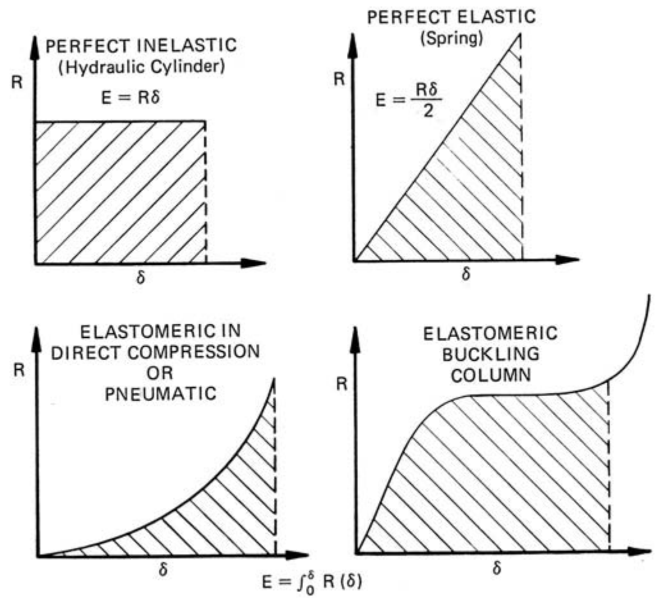

Fig. 5-12. Fender force–deflection relationships

For example, for the linear elastic case of a steel spring or cantilever pile, the energy is equal to one-half the reaction (R) times the deflection (δ). A spring constant, $K_F = R/\delta$, can be defined over the elastic range of the spring, and therefore,

$$R = \sqrt{2KE} \tag{5-16}$$

When the pier or dolphin deflection is significant, the fender and structure act like springs in series, and the combined spring constant (K_c) can be found from

$$\frac{1}{K_c} = \frac{1}{K_F} + \frac{1}{K_s} \tag{5-17}$$

where K_s is the equivalent spring constant of the structure. This equation can be used for any number of springs in series by summing the reciprocals of the linear spring constants. For springs in parallel, such as for a rigid wale backed by several units along its length, the spring constants are directly additive; that is, $K_c = K_{F1} + K_{F2} + \cdots + K_{FN}$. For a hydraulic-type fender, the load remains almost constant with deflection, so that the maximum energy is extracted per unit deflection, and the fender does not recoil, reflecting energy back into the ship. For most compressible and buckling column-type resilient rubber units in common use today, however, the spring constant cannot be defined, except along a limited length of the curve. Therefore, common practice is to obtain both the reaction and the energy at a given deflection from the manufacturer's published data.

Care must be taken to correct published energy values for the angle of contact. Angle of contact and angle of approach, or berthing angle, are not necessarily the same. Only when berthing against dolphins that are close enough together that contact is made only on the straight side of a vessel, as in most oil terminals, are the berthing angle and the contact angle the same. Along a continuous dock line, Eq. (5-18) gives the absolute maximum contact angle, α, which can occur independent of the actual berthing angle of the vessel's centerline:

$$\sin \alpha = P/2r \tag{5-18}$$

where
P = fender pitch, or distance between consecutive fender centerlines, and
r = vessel hull radius.

The effect of angle contact on fender performance is purely a matter of geometry. The ratio of a fender element's height, or standoff, to the overall width of all the rubber in a given fender determines the severity of the effect of angular deflection. When rubber is arrayed horizontally on a dock fascia and a vessel makes angle contact, only one side of the rubber is fully deflected. The part of the rubber that is not deflected cannot absorb energy. If a fender could be made infinitely narrow, so that all its effort were concentrated at its centerline, it would suffer no angular-contact effects. Perhaps the simplest example of the fact that it is geometry that affects performance, not the angle of contact per se, is the typical arch, or V-shaped, fender. When such a fender is used with its long axis vertical, the available energy is normally 90% to 99%, depending on angle. When the same fender is used with its long axis horizontal, the available energy is often reduced by 50% to 90%, depending on angle and length. Thus, in most cases it is generally recommended that fender energy-absorbing elements be concentrated in as narrow an area as possible.

Fender Types

One type of fender system that is less common today is the traditional rigid timber framework and/or a row of piles interconnected by timber wales that absorb energy through the compression and bending of the wood fiber. These systems have a minimal energy-absorption capability without permanent damage and sometimes are backed up at deck level by hollow-core rubber units. Fig. 5-13 illustrates a typical timber fender pile system, and Fig. 5-14 shows a continuous timber fender pile rack traditionally used in ferry slips. Timber systems usually are economical on a first-cost basis and adaptable to large tide ranges. Their disadvantages include high maintenance costs and limited protection. Derucher and Heins (1979) provide some design guidance. Many timber systems are being gradually replaced by more contemporary rubber units. Bradshaw et al. (2006) developed a simple dynamic model for fender pile analysis that demonstrates up to 25% reduction of reaction forces compared to the traditional approach of equating vessel kinetic energy to pile system force and

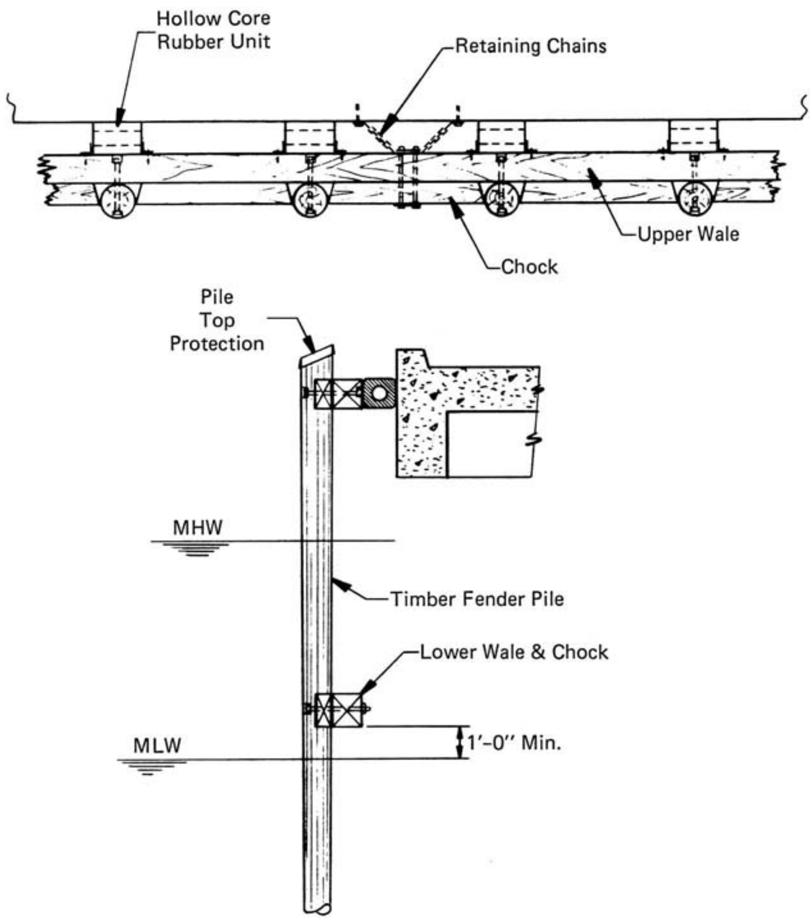

Fig. 5-13. Timber fender pile system

deflection. They field-tested their model results on an instrumented composite fiber-reinforced plastic (FRP) pile, which may be a viable option to timber piles at many sites.

Camels are floating devices that are used to increase vessel standoff distance and to distribute berthing and mooring loads along the pier face, especially where fender pile systems are used. They may simply consist of a log or pile tethered to the pier or quay face, they may be more substantial steel pipe and composite cylindrical structures, or they may be larger and much wider built-up timber, concrete, or steel pontoon structures (see Section 9.1). Camels may be fitted with fenders and constitute an integral part of fender systems but are not necessarily energy absorbers. Caution should be exercised when using camels with springing pile fenders because they may exert a concentrated load at the piles' midspan, resulting in frequent pile breakage. Therefore, camels used in this way should be long enough to distribute berthing and mooring loads over a sufficient number of piles. Submarines require

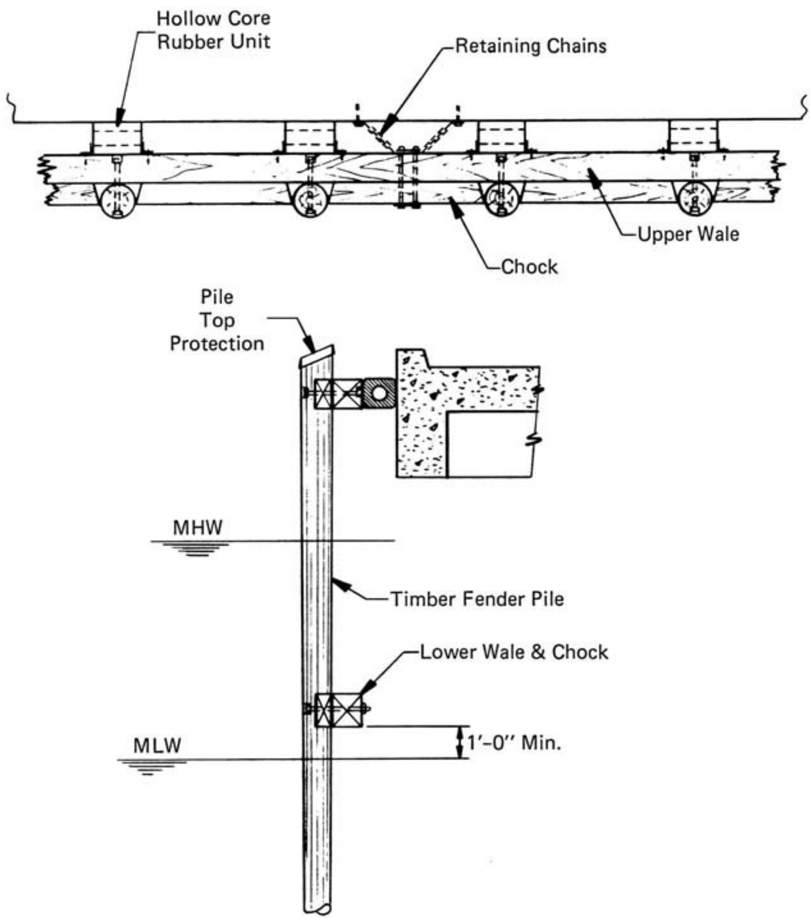

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Fig. 5-15. Pneumatic-type floating fenders alongside pier

Source: Photo courtesy of Seaward International, Inc.

of a closed-cell foam flotation material encapsulated in a polyurethane shell with a continuous tension member, usually chain, along its central axis connecting mounting eyes at each end. They typically float with a draft of between 20% and 40% of their nominal diameter and, similar to pneumatic-type fenders, require an ample backup face area to accommodate their compressed, flattened shape. They may also be provided with chain tire nets similar to pneumatic types, and both types may also be used as separators between vessels and are available in a large range of sizes and energy-absorption capacities. Pneumatic and foam-filled-type fenders may alternatively be suspension mounted on the quay face as fixed, nonfloating fender units.

Hanging or draped fender units consisting of hollow-core rubber sections sometimes are used at solid-face quays.

Rotating fenders typically consist of either a pneumatic tire or a foam-filled doughnut rubber unit mounted on a vertical shaft. The tire or cylinder is free to rotate, thus reducing friction, and to absorb energy by its deformation. Such fenders are ideally suited to applications where vessels frequently are warped about them. The pneumatic tire types are primarily used as guide-in fenders at the entrances to locks and dry docks. They are generally less tolerant of vertical motions.

Considered mostly obsolete today are mechanical-type systems consisting of springs and linkages, and/or gravity systems consisting of suspended weights or buoyancy units.

Hydraulic fenders may be preset to a given reaction-force level. Bruun (1984) has advocated their use at exposed locations because of their energy absorption efficiency and nonrecoiling characteristics. The fact that they do not immediately return to their original position after impact may be a problem at certain locations. Others have noted that hydraulic units are likely to require greater than average maintenance (PIANC 1984). *Hydro-pneumatic fenders* are partially filled with air and water and are used where fender contact is required below the waterline, such as for submarines or semisubmersible drill rigs.

At most major marine terminals today, high-energy-capacity *resilient rubber units* are used. There is a wide variety of shapes and sizes of elastomeric units on the market today that can be of molded, extruded, or wrapped-type manufacture. Buckling column type, including cells, delta, and unit types, typically reach their maximum rated reaction force at around 30% compression and maximum rated energy at 50% to 70% compression. This is important to keep in mind when selecting a fender based on rated energy capacity because the pier structure is exposed to the full fender reaction at much lower impact energies. A representative straight-leg buckling column fender (Fig. 5-16), for example, reaches maximum

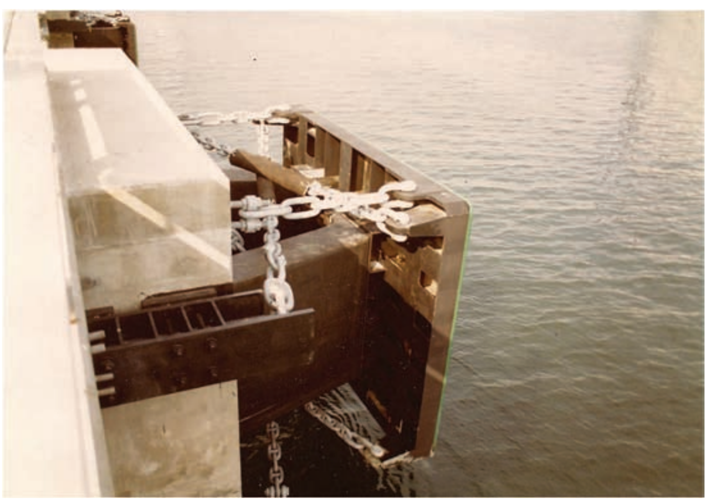


Fig. 5-16. Dual single-element fender units mounted vertically with fender face panel and retention chains

rated energy capacity at 57.5% compression and is fully compressed at 62.5% compression, beyond which the reaction force goes asymptotic if all the berthing energy has not been absorbed. Furthermore, if the vessel contacts multiple fenders such that the fenders absorb the design-level energy without buckling, then the total force on the pier or berthing structure may be greater than the rated reaction at capacity. Fig. 5-17 illustrates some of the more common generic fender types. The elastomer is of either a natural rubber or a synthetic rubber, the properties of which can be varied to obtain different characteristics (see Section 5.7). Rubber units may be worked in direct (bending) compression of hollow cross sections, bending via buckling-column action or in shear or torsion. Shear fenders are more difficult to construct and are more likely to break than other rubber fender types.

Parallel motion fenders are of more complex design, with mechanical linkages that provide extremely low reaction forces for a given energy and no performance loss at large angles (Fig. 5-18). These fenders are typically custom designed by a qualified fender manufacturer. They may be cost-effective, especially where low reaction forces are required because of berthing structure limitations. Beckett (2010) provides a detailed discussion of important design considerations.

Monopiles are simply cantilevered piles that serve as fender dolphins where subsurface soil conditions permit (see Section 7.5). They may themselves be fitted with rigid or resilient fendering and may be interconnected with other piles to form *berthing beams*. Steel pipe pile dolphins have also been fitted with interconnecting torsion bars to extract additional energy from the pipe in torsion.

Padron and Han (1983) present the results of a study of fender system problems in U.S. ports based upon a study conducted for MARAD (Han et al. 1978) that should be of interest to those evaluating alternative fender types with particular regard to maintenance costs and problems. Their findings are summarized in Table 5-2, and the most prevalent types are illustrated in Fig. 5-19. Padron and Han found that timber systems generally had greater maintenance problems than rubber systems; chief among these problems is damage caused by high-energy berthings, wear, and attack by marine organisms. Rubber fender units, when properly sized and installed, may have practical design lives of 15 to 20 years or more, depending upon the level of activity at the berth (Thoresen 2014, Piaseckyj 1977). Replacement of deteriorated systems with contemporary resilient fenders can play an important role in the upgrading of existing facilities (Asayama and Ohtsuka 1983).

5.6 Fender System Layout and Design

The design of marine fender systems usually begins with a determination of the fender energy absorption requirements and allowable reaction, and moves on to ascertaining if there are any standoff restrictions. If there are no standoff restrictions, there is usually a choice of multiple fender types and sizes that can be used. Some types may prove impractical, and some may be too costly. If there are maximum

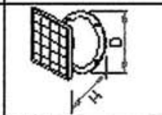
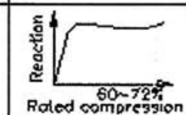
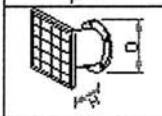
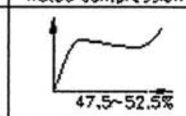
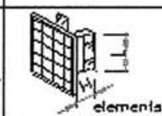
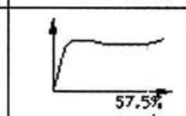
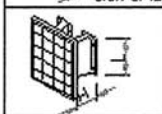
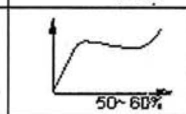
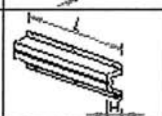
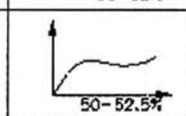
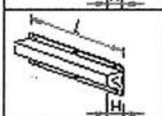
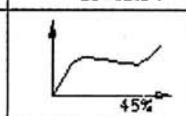
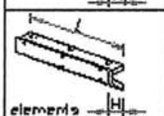
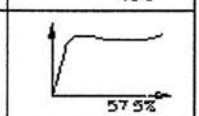
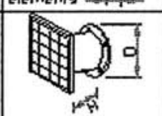
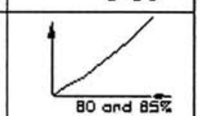
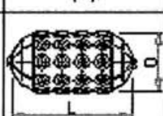
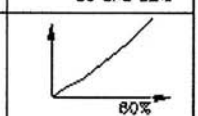
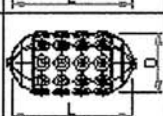
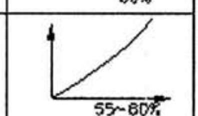
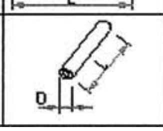
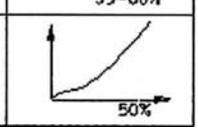
	Type	Fender shape	Sizes in mm	Reaction kN	Energy kNm	Performance curve
Bucking type fender	Circular shape of the bucking fender with panel contact		D/H 500/300 ↓ 3200/2000	60 ↓ 4660	9 ↓ 4840	
			D/H 650/400 ↓ 3350/3000	56 ↓ 5688	10 ↓ 6570	
	Longitudial shape of the bucking fender with panel contact		H/L 300/600 ↓ 1800/2000	66 ↓ 1708	9 ↓ 1260	
			H/L 400/500 ↓ 2500/4000	140 ↓ 6900	22 ↓ 7000	
	Bucking fender with direct contact		H/L 250/1000 ↓ 1000/2000	150 ↓ 2290	15 ↓ 940	
			H/L 200/1000 ↓ 1300/3500	150 ↓ 3400	10 ↓ 1500	
			H/L 300/600 ↓ 1000/2000	45 ↓ 646	6 ↓ 297	
Pneumatic	Airblock		D/H 600/450 ↓ 3200/3200	138 ↓ 6210	15 ↓ 4990	
	Pneumatic		D/L 500/1000 ↓ 4500/12000	50 ↓ 10570	4 ↓ 9080	
	Foam filled		D/L 1000/1500 ↓ 3500/8000	200 ↓ 4050	41 ↓ 3000	
Side loaded	Cylindrical		D/L 150/1000 ↓ 2800/5800	80 ↓ 6600	3 ↓ 5000	

Fig. 5-17. Common generic fender types and their characteristics

Source: Guidelines for the Design of Fenders Systems: 2002, Report of WG 33, MarCom, PIANC; reproduced courtesy of PIANC

Fig. 5-18. Parallel motion-type fenders with back-to-back mounted cones at an open pier structure supporting a conveyor system

Source: Photo courtesy of Trelleborg Marine Systems

standoff limitations, some types of fenders may be incapable of meeting the energy and reaction requirements with the restricted deflection space available.

When maximum standoff is limited, buckling-type fender elements are often the most favorable type capable of absorbing the necessary energy without exceeding the permitted reaction. This is because of the greater area (energy absorption) under their reaction/deflection curves.

Other fender type selection factors are introduced in Section 5.5, and Table 5-3, from BSI (1994), summarizes additional design considerations for specific facility types.

The need to consider a fender's performance while the vessel is moored cannot be overemphasized. This is especially true at exposed locations and for larger vessels subject to dynamic forces (Brunn 1984, Khanna and Birt 1977). The fender load/deflection properties should be compatible with the elasticity of the mooring lines at open-sea berths (Koman 1980), and berthing/mooring design requirements should be integrated. Vessel movements in ports while moored can be greatly influenced by the fender system and in turn exert forces and wear and tear on the fender system (see Sections 6.1 and 6.11).

Since most modern fenders currently use rubber energy-absorbing elements, it is important to understand that all rubber devices exhibit a characteristic called *hysteresis*. This might better be thought of as intermolecular friction. This characteristic is strain-rate dependent. Therefore, the performance of rubber devices, even within a linear region of a reaction/deflection curve, cannot be modeled perfectly correctly with a simple spring constant. Because of hysteresis, rubber devices incorporate both a deflection-based (spring) component and a deflection-rate-based

Table 5-2. Ranking of Fender System Problems and Fender System Types by Prevalence/Severity of Fender System Problems

Description of Fender System Problem	Rating of Severity of Fender System Problems as Percentage of Total Wharf Length												Rank of Fender System Problem
	Timber Fender Systems							Rubber Fender Systems					
	A34.3	B9.1	C4.9	D3.3	E5.9	F1.2	G14.1	H22.9	I2.1	J1.7	K0.5	Summation	
High berthing energy	5	4	5	5	5	4	5	1	1	1	1	380.9	1
Wear	4	3	5	5	3	3	3	1	1	2	2	298.5	2
Deterioration by marine organisms	4	4	4	4	4	4	2	1	1	3	1	293.6	3
Securing lines to fender system	4	4	4	1	3	3	—	—	—	—	—	217.8	4
Performance adversely affected	2	2	2	2	3	3	3	1	1	2	1	195.7	5
Snagging by vessels	1	1	1	1	3	3	3	1	3	3	3	151.0	6
Corrosion of steel components	1	1	1	1	1	1	1	2	2	3	3	129.4	7
Summation	720.3	172.9	107.8	62.7	129.8	25.2	239.7	160.3	18.9	23.8	5.5		
Rank of fender system type	1	3	6	7	5	8	2	4	10	9	11		

Note: Summations are the sum of the products of wharf length and rating for each row or column.
Source: Padron and Han (1983); copyright ASCE.

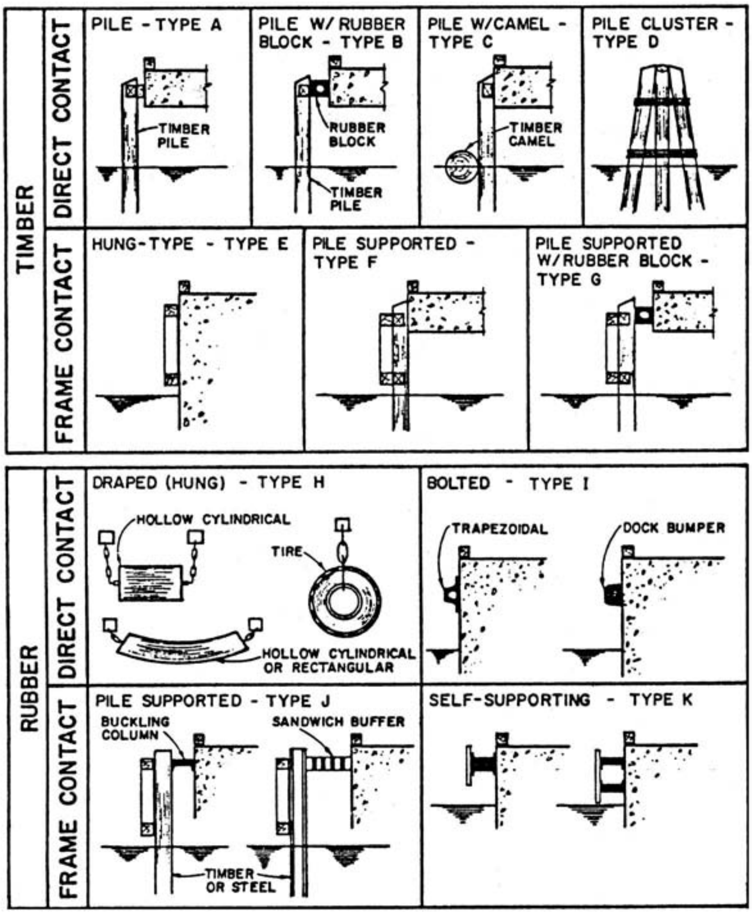


Fig. 5-19. Prevalent types of timber and rubber fender systems

Source: Padron and Han (1983); copyright ASCE

(damper) component (i.e., *viscoelastic* behavior). The net result is that not only is deflection reaction increased in response to strain rate, but also recoil reaction is decreased in response to the rate of recoil. Depending on the rubber compound, this difference can be as much as 50% or more. Because there is virtually no information in print giving rubber fender reaction versus recoil deflection characteristics, caution should be exercised when attempting to incorporate fender performance into mooring simulations. Further information on variables affecting rubber fender performance are reviewed in the following section.

Some engineers recommend that the ultimate capacity of a fender unit and its supporting structure should be on the order of twice the nominal design energy level, subject to site-specific studies, which may determine that a higher or lower figure is warranted (BSI 1994). However, common practice over the past 15 years in

Table 5-3. Facility Type Fender Design Considerations

Vessel Facility Type	Fender System Design Considerations
Tankers	Large change in draft loaded to empty Minimize standoff distance Fire hazard from friction and/or sparking Bulbous bow Small or coastal tankers may have minimal freeboard and berth without tugs, manifolds not at centerline
Gas carriers (LNG/LPG)	Shallow draft with small draft change loaded to empty Low allowable hull pressure Berth often dedicated to single vessel type Fire hazard from friction and/or sparking Bulbous bow
Bulk carriers	Large change in draft loaded to empty Minimize standoff distance Low allowable hull pressure May need to be repositioned or warped in berth Bulbous bow
Containerships	Minimize standoff distance and crane outreach Bow flare overhang Low allowable hull pressures for larger ships Bulbous bow
General cargo ships	Minimize standoff distance Relatively large draft change loaded to empty Smaller or coastal vessels may berth without tugs
Ro/Ro vessels	Loading ramps, side, end, and slewing High berthing velocities End-on berthing Hull belting with variable shapes and sizes High freeboard with poor maneuverability Bulbous bow
Ferries	Quick turnaround and frequent berthing High berthing velocities End-on berthing Hull belting
Passenger and cruise ships	High freeboard, bow flare, and flying bridge overhang Little draft change loaded to empty Boarding ramps at various locations Low allowable hull pressures Berth without tugs Bulbous bow
Miscellaneous vessel types	Service craft, tugs, fishing boats, barges, military, and high-speed vessels may all have specific fendering requirements. See vessel types discussion in Section 2.4

Source: This table is compiled in part and adapted from data in BSI (1994) and Trelleborg AB, *Safe Berthing and Mooring*, Product Catalog (2008).

the United States and Canada when using rubber fenders has been to design fenders to meet design energy requirements without exceeding allowable reaction and without any further explicit factor of safety. The design energy should be based on an accurate determination of design berthing velocity, which should correspond to a 98% to 99% berthing velocity confidence limit.

Design berthing velocity and the normal, most usual, berthing velocity varies widely depending on berthing frequency, exposure, and known hazards. For instance, from a purely engineering design standpoint, high-frequency ferries should have design berthing velocities that are perhaps four times normal, whereas oil tankers should have greater margins of safety than ferries, even with design berthing velocities that are equal to their normal berthing velocities. This is because with the schedule sensitivity of high-frequency ferries, abnormal berthings occur on a more frequent basis. Obviously, selecting an appropriate design berthing energy is a highly subjective exercise, and one in which an inexperienced designer may want to ask for recommendations from fender manufacturers.

Any overload factors depend to some degree upon the type of fender system, its mode of failure, and the consequences of such a failure. Steel pipe pile dolphins, for example, which absorb energy via bending deflection, often are designed to work at up to 80% of the yield stress in the steel under nominal design conditions, corresponding to a factor of safety of approximately 1.5 on the design energy at the yield of the steel (PIANC 2002). This example illustrates an important point. The factor of safety in this example may at first have seemed to be 1.25, the ratio of the design yield stress to the actual yield stress. However, in fenders, the controlled variable is energy, not stress. So the comparison is actually between the energy that can be absorbed with a stress that is 0.8 of yield and the energy that can be absorbed at yield. Since energy is the product of reaction times deflection, and since reaction in this example is proportional to deflection, the energy absorbed is actually proportional to the square of the applied stress. Thus, the theoretical factor of safety actually calculates to 1.25 squared, which is 1.56. This is a more appropriate method to calculate actual factors of safety. Additional discussion of monopile and other constructed-type fender dolphins is provided in Section 7.5.

Other aspects of fender system design that are discussed further here include vertical and longitudinal rubbing forces, face-panel dimensions with regard to allowable hull pressures, dimensioning, layout and spacing requirements, and material specifications. Worked examples of the design of various types of fender systems may be found in the product literature of certain fender manufacturers.

When doing preliminary evaluation of various alternatives before settling on a preliminary fender design, it can be helpful to use some of the following rules of thumb [provided by Ed Kiedaisch in MMI (2002)].

- E and R are directly proportional to rubber stiffness. If one fender has a rubber compound 15% stiffer than another that is otherwise identical, both its E and R are 15% greater than the softer fender.

- E and R vary directly with fender-element length. If two fenders are otherwise identical, a 1,500-mm-long fender has 150% the E and R of a 1,000-mm-long fender.
- Rated R is directly proportional to H for a given rubber compound and fender-element length.
- Rated E is proportional to the square of H for a given rubber compound.
- Rated E is proportional to rubber volume. Any two fenders of the same design and rubber compound have rated Es in direct proportion to their volumes of rubber, regardless of differences in element heights and/or lengths.
- Fender element cost varies (approximately) directly with rubber volume. Costs per unit volume are usually very close for fender heights within two sizes of each other, regardless of rubber compound.
- Fender efficiency (E/R, not R/E) varies directly with H. All fender elements of the same H have the same E/R. Compound and length have virtually no effect.
- Use a fender with E/R greater than or equal to the specified E divided by the specified R. No fender with a lower E/R can meet the specification. Usually, rated E/R needs to be 5% to 15% greater than the nominal specification to allow for the effects of angle berthing and bow flare.
- Contact panel area varies inversely with hull pressure.
- Contact panel cost usually varies (roughly) directly with overall panel area. Panel area is based on overall dimensions, including any bevels on the top, bottom, or sides.
- For barge or ferry service, increase ultrahigh-molecular-weight polyethylene (UHMW) thickness by 1/4 to 1/2 in. (6 to 12 mm).

Where loss of a single fender for even a few hours can have serious financial or political consequences, fenders should be designed for easy repair and/or removal/installation. A good example would be a rubber buckling-type fender with panel and weight restraint that can be removed or installed without a wrench, requiring only removal of a locking pin and lifting (or the reverse when installing).

In the vast majority of instances, fenders should be designed with the understanding that they from time to time become hooked by hull belts as vessels rise and fall because of draft and/or tidal changes. With proper maintenance, most fenders can be maintenance-free for 10 years or more. Many properly designed fenders seldom require maintenance.

Mounting and Installation

Fender installations should be as simple and rugged as possible. The use of chains or other flexible restraints, turnbuckles, and moving parts should be kept to a minimum. The ability of a pier or berthing structure to distribute and resist all berthing and mooring reaction forces must be carefully checked. Fig. 5-20 shows a representative contemporary rubber, buckling-column type fender installation fitted with a

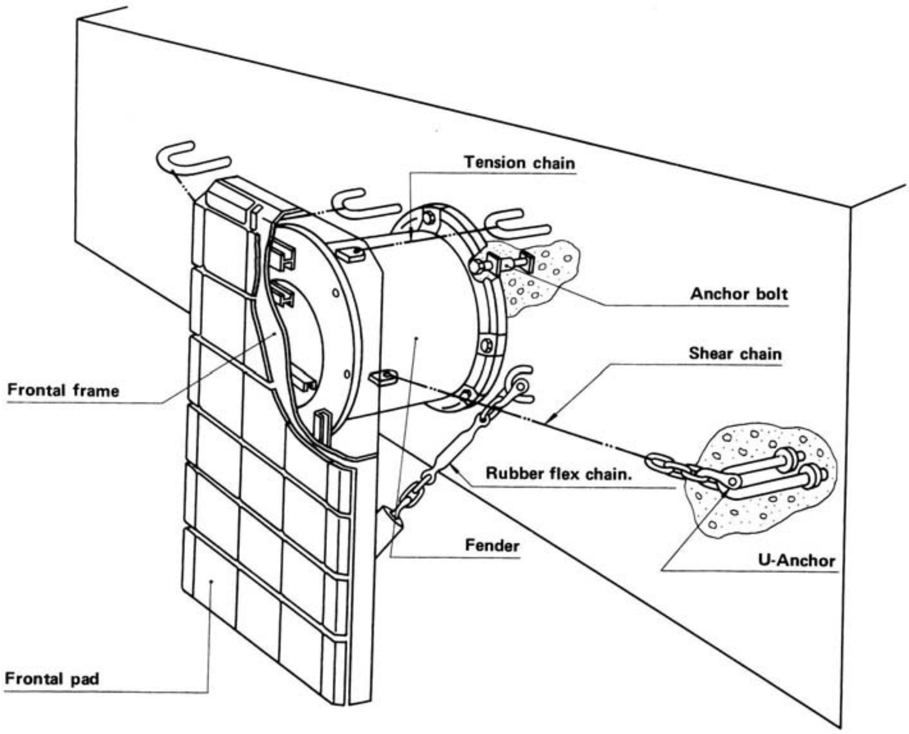


Fig. 5-20. Rubber fender installation

Source: Bridgestone Tire Co. (n.d.); reproduced with permission from Bridgestone

low-friction face panel and flexible restraints. Alternatively, the face panel may be mounted on piles where large tide ranges must be accommodated. The installation of high-energy-capacity units on piers often requires the use of raised parapets or lowered skirt beams along the pier face.

Chains are often an integral part of fender systems provided for preventing fender damage by relieving weight and checking excessive movements and for retention of floating fenders, for example. Chains can therefore be broadly categorized as follows:

- Weight or suspension chains,
- Tension chains,
- Shear chains, and
- Retention chains.

Guidance for designing chains and their arrangements can usually be found in a manufacturer's product literature. Allowance for corrosion and wear and means for ready replacement of damaged chains are important considerations.

Rubber fenders generally are capable of very large shear deflections without damage, and under the design loads there is likely to be no damage caused by the deflections. Rubber dock fenders that are self-supporting and have a frontal load-distribution panel often use suspension, or weight, restraints to help support panel weight, and sometimes also use shear and tension restraints for improved performance. The use of weight restraints is not so much to limit creep, which it does, but to improve fender useful life. When weight restraints are used, they are best attached to the panel at or slightly above the vertical center of the rubber elements' reaction. Since most flexible restraints are incapable of absorbing significant energy, they are not useful to restrain "hooking." Shear restraints are useful in applications in which vessels may use the fender for warping, and they may also be useful in oil tanker unloading berths. Vertical shear restraints are often useful in oil tanker unloading berths. Tension restraints often are required when low-freeboard vessels contact fender panels below the center of the fender elements' reaction.

Rubbing Forces

Because the velocity of impact seldom acts exactly normal to the fender face, there usually is some horizontal component of velocity acting parallel to the face of the structure. If shear friction is a concern, either because of the distortion it induces in the fender or because of the wear it causes in fender contact surfaces, fender contact surfaces should be of ultrahigh-molecular-weight polyethylene (UHMW).

The actual dry coefficient of friction of UHMW against steel is less than 0.2. However, it is not very hard, and very high mooring or storm forces can cause imprinting of hull irregularities into the surface of the material, raising the effective coefficient of friction. For most applications, the use of 0.3 as a design coefficient of friction is a conservative value. For unusual, high-pressure applications, contact a fender manufacturer. There is a wide range of reported values of the coefficient of friction against a steel hull, ranging from 0.3 to 1.0 for timber (0.4 to 0.6 is more typical), 0.15 to 0.75 for steel (0.25 is more typical), 0.5 to 1.0 or more for rubber, and 0.08 to 0.2 for UHMW. The actual value also depends upon whether the surfaces are wet or dry, covered with ice or oil, rough or smooth, and the condition of the vessel's hull.

Maximum rubbing forces, both vertical and horizontal, usually are associated with mooring forces rather than berthing forces, although it is usual for designers to apply an assumed value of μ times the design berthing reaction force. Vertical forces, however, are more likely to occur while the vessel is moored than during berthing. DOD (2005) suggests designing for a longitudinal force of 0.5 times the maximum berthing load and for a vertical force of 0.3 times the maximum berthing load. A value of 0.3 for both vertical and horizontal shear loads seems reasonable for most design applications. The primary application that clearly exceeds these values from time to time is oil tanker unloading berths, and this exceedance occurs entirely because, at many terminals, ships' crews can be negligent in tending lines. As the tankers are unloaded, they may rise more than 20 ft. If lines are not carefully tended, this results in the vessels being pulled tightly against the fenders. Strong upward vertical shear restraints are not always successful in preventing damage under these conditions. It is recommended that the vertical and longitudinal rubbing forces associated with the fender reaction forces under the maximum design mooring conditions also be checked and compared with the berthing conditions.

Allowable Hull Pressures and Fender Face Dimensions

Vessel parameters affecting fender layout and design include hull strength and allowable bearing pressures and hull geometry, especially hull curvature and length of parallel midbody. For smaller vessels, the local hull strength is not usually a problem because of the closer frame spacing, greater curvature, and inherently greater stiffness, compared to larger vessels. For larger vessels with large areas of vertical and parallel sides, the shell plating and stiffeners, which are designed for local hydrostatic pressures, are vulnerable to local point loads. The allowable hull pressure for a given vessel then depends not only upon the fender face contact area, but also, importantly, on its placement in relation to the ship's shell plating and internal framing (see Section 2.3).

Svendsen and Jensen (1970) provide a general solution in graphical form for tankers and bulk carriers based on the plastic moment capacity in bending of longitudinal and transverse stiffeners and frames for mild ship-hull steel. The ratio between the fender panel width and the vessel transverse frame spacing should not be less than 0.5 to 0.65, and the ratio between the fender panel height and the side longitudinal spacing not less than about 2. Ideally, the allowable pressure should be obtained from the vessel's owner/operator and its naval architect. In lieu of specific vessel information, PIANC (2002) provides some general guidance summarized in part as follows:

- LNG/LPG and contemporary bulk carriers: <4,200 lb/ft^2
- Tankers: < 60,000 DWT: <6,200 lb/ft^2
- Tankers: Very large crude carriers (VLCCs): 3,000 to 4,200 lb/ft^2
- General cargo >20,000 DWT: < 8,400 lb/ft^2
- Containerships: First and second generation (<1,000 TEU): <8,400 lb/ft^2
- Containerships: Panamax (<3,000 TEU): <6,200 lb/ft^2
- Containerships: Super post-Panamax (>8,000 TEU): <4,200 lb/ft^2

The above tabulation has been abbreviated from the PIANC table, and values have been converted from SI and rounded to U.S. customary units. Values from PIANC (2002) go as high as 14,600 lb/ft^2 for smaller general cargo vessels. Thus, the above values fall in the range of <30 lb/in.^{2} (4,320 lb/ft^2) to approximately 100 lb/in.^{2} (14,400 lb/ft^2) maximum. Note that many vessel types, such as Ro/Ro and ferries, are typically "belted" with steel strakes projecting typically from 8 to 16 in.

and of similar width girdling the vessel at the main deck level and sometimes at upper decks as well. Barges and some tankers, as well as tugs and workboats, often have belting of half-round shape that produces high local contact pressures. Although the belting protects the vessel, it produces high localized loads and wear on fender face materials that must be considered in fender system design. In addition, belting may become caught on the tops or bottoms of fender face panels with changing water levels and vessel drafts so that careful attention to vessel positions and face panel design is required.

Caution should be applied in dealing with lightly constructed vessels, such as certain contemporary naval warships; NFESC (1997) gives limiting hull-contact pressures for uniform contact over an entire shell plating panel area ranging from as low as 8 lb/in.^{2} up to 87 lb/in.^{2}; the majority of U.S. Navy vessel types are more within the range of 15 to 20 lb/in.^{2} (2,160 to 2,880 lb/ft^2) for a wide range of vessel types. The NFESC technical report (1997) also gives limiting pressures and total forces based upon the fender load distribution area in relation to the ship's framing. Where hull pressures may be critical, the naval architect or vessel owners should be consulted for specific requirements.

Fender Spacing and Layout

The vessel's geometry affects fender spacing in particular, as well as the location and number of fenders contacted. Fender spacing additionally depends upon the type of fender system and structural support, the range of vessel sizes to be accommodated, and the type and arrangement of berth and mooring loads. Typical berth arrangements are discussed in Chapters 3 and 6. A tanker often is moored against two discrete dolphins, spaced on the order of 25% to 50% of the vessel's LOA apart. The fender spacing should allow for the smallest design vessel to safely lie alongside at any location. Spacing ranging from 8% to 15% BSI (1994) of the vessel's LOA have been proposed. The distance between centers of fender units often is referred to as the *pitch*. Specific recommendations have been made for Ro/Ro and ferry-type vessels (PIANC 2002, BSI 2014). Fig. 5-21 illustrates typical fender spacing for various berth types.

As a guide to establishing distance between fenders on a continuous wharf, the formula below gives an indication of the approximate, safe maximum fender pitch, P, for the smallest ship's bow radius (r):

$$\mathrm{P} = 2\sqrt{r^2 - [r - h - C]^2} \qquad (5\text{-}19)$$

where

r = radius of hull curvature at level and point of contact;

h = overall fender standoff at rated deflection, measured on the fender centerline; and

C = desired clearance between vessel at point of closest approach and dock fascia (should be at least 10% of undeflected fender standoff).

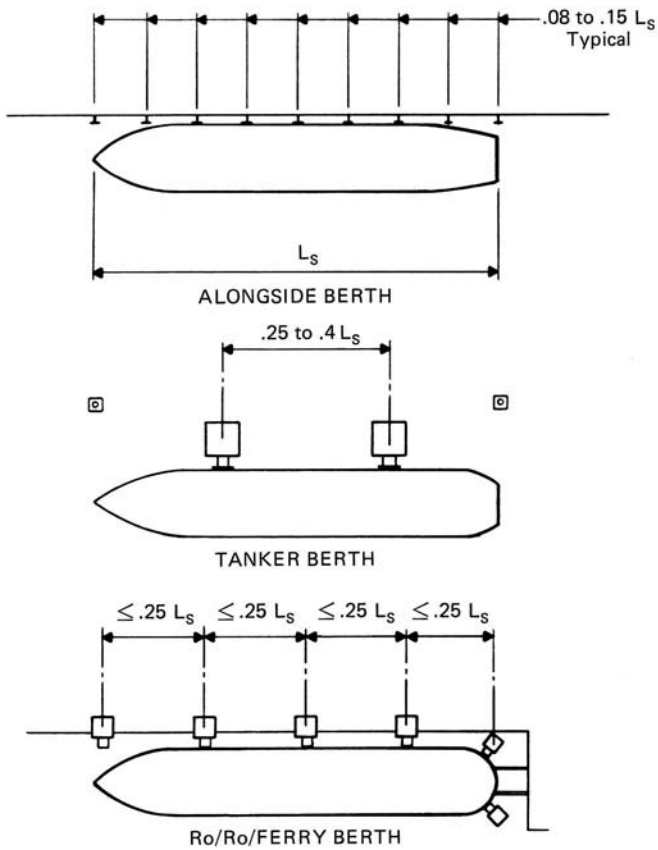


Fig. 5-21. Fender spacing for typical berth arrangements

Source: Compiled from data from BSI (1994)

The pitch produced by this formula is that which will not permit the vessel with the hull radius used for the calculation to approach any closer to the dock fascia, at a point halfway between adjacent fenders, than the clearance value inserted in the formula. All vessels with hull radii greater than the value used in the calculation have even greater clearances at full fender deflection. Technically, a vessel alongside only requires two points of contact while in berth, although three or more are recommended, which means that the absolute maximum spacing is controlled by the length of the vessel's parallel sides. However, when berthing alongside a pier, any fender pitch greater than that calculated above requires that care be taken to ensure that berthing vessels align themselves correctly with the fenders before attempting to berth against them. In general, the ratio of a vessel's parallel midbody length to its overall length is on the order of 35% to 55% of its LOA, usually larger for longer vessels. This ratio often determines the point of first

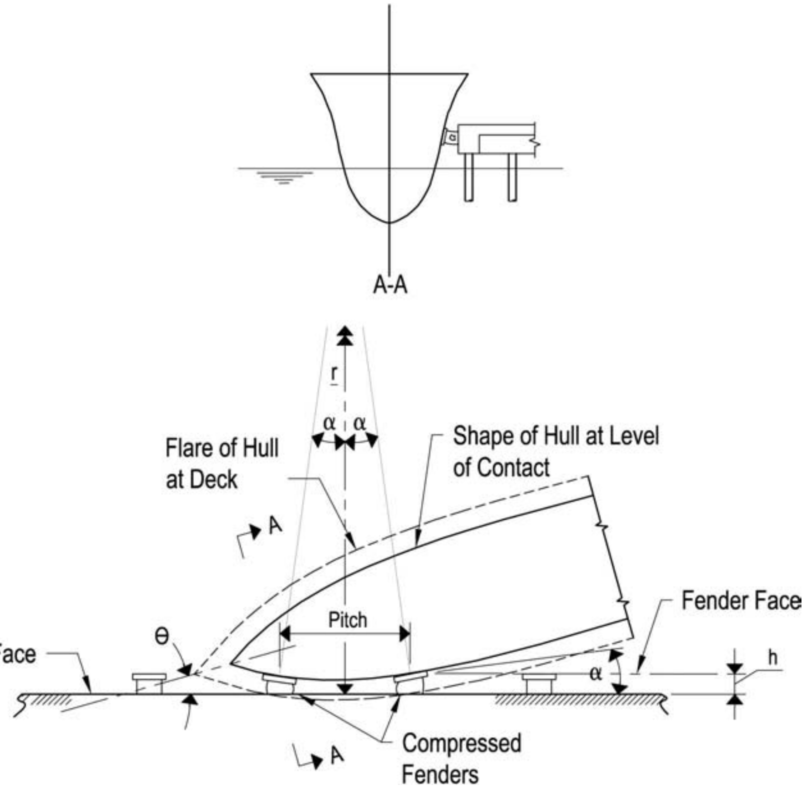


Fig. 5-22. Effect of hull shape on fender spacing

contact with the vessel's hull, which is usually at the end of the parallel midbody, and also the length of vessel available to contact fenders under moored conditions.

The effect of the hull's curvature near the bow or stern on fender spacing is illustrated in Fig. 5-22. Vertical curvature of the hull, hull flare, and overhangs and/or projections, such as bulbous bows, also must be considered in fender system layout. As discussed previously, greater standoffs can often lead to more economical fender designs. However, in the case of containership and dry bulk berths, the maximum undeflected standoff must not exceed the maximum permitted by the berth's cranes or loading/unloading equipment. The minimum standoff at 120% of rated energy, or a fully compressed fender, should not permit contact between the berthing vessel superstructure and wharfside equipment, or between the bottom of the hull and battered piles or the dredged slope at the bottom of the berth. The standoff distance usually ranges from 3 to 6 ft for most seagoing terminal facilities. At offshore installations, this distance may be closer to 10 ft or more. Note that for the two-fender contact shown in Fig. 5-22, the berthing energy is shared by two fenders, but given the fact that buckling-type fenders reach their maximum reaction force at

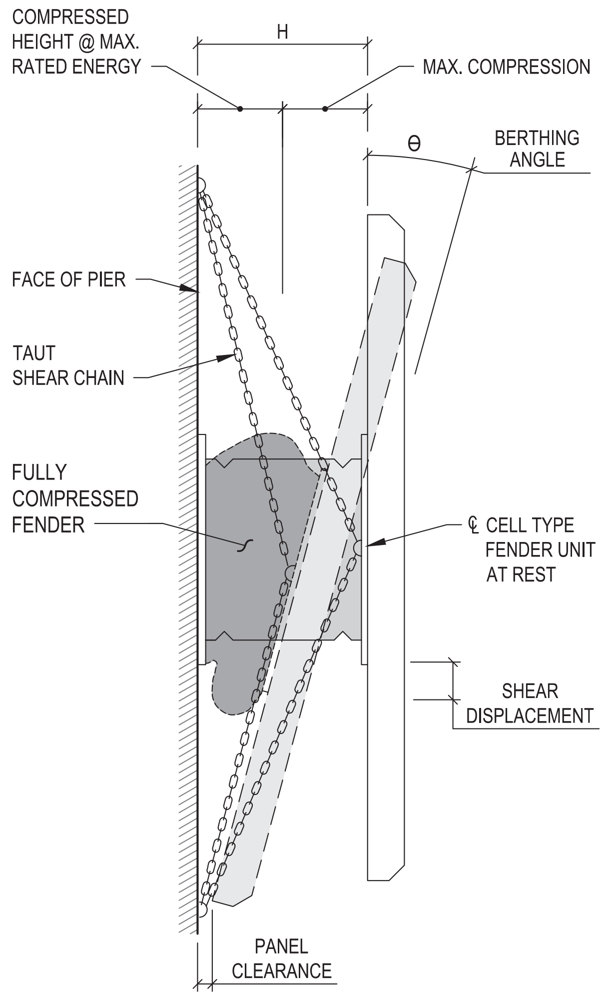


Fig. 5-23. Deflected geometry of fender face panel under combined shear and compression

about 30% deflection, they may exert greater total force on the pier or wharf than a single fully buckled fender for the same berthing energy. For multiple fender contacts, the energy is shared among the fenders, but they are not necessarily shared equally. Such would be the case for a three-fender contact, where the central fender would have a greater deflection than the adjacent fenders. The designer must therefore consider the load deflection characteristics as well as the vessel geometry to determine the total force and load distribution along the pier structure. For angular impacts, the fully compressed fender also deflects horizontally until checked by the shear chains, and the face panel assumes a new rotated position, as illustrated in Fig. 5-23, which also needs to be checked in the design process.

5.7 Fender Materials and Specifications

Hardware

Fender system hardware includes shackles, chain hardware, bolts, and connection hardware, as well as anchor bolts and pad eyes. All materials for fender installations should be specified to meet minimum strength and durability requirements, as described in Section 3.5. All metal hardware items should in general be galvanized, per the requirements of either ASTM A153 or B695 or should be of AISI 316 stainless steel. Some fender manufacturers also offer protective coatings, which may require recoating on a more frequent basis than would a touch-up of galvanizing. Anchor bolts are normally called out under ASTM F-1554-GR 105.

Rubber

Fender rubber compounds may be of natural rubber (NR) or of synthetic rubber elastomers, such as styrene butadiene rubber (SBR), butyl rubber, neoprene, ethylene propylene diene monomer (EPDM), and polyurethane. Rubber properties can be varied widely to obtain the desired stiffness and resistance to aging and atmospheric ozone. The final rubber compound is a blend of raw rubber, a reinforcing filler called carbon black, and other chemicals that improve its physical properties. As previously discussed, an important property of elastomeric materials is their hysteresis. Rubbers with a greater hysteresis exhibit less recoil or tendency to bounce the vessel off. Rubber exhibits a unique rheological response under stress in that its reaction force and energy absorption both increase with the rate of load application or strain rate. Rubber's elastic properties also vary significantly with temperature, generally becoming stiffer at lower temperatures. Therefore, a velocity factor (VF) and temperature factor (TF) correction needs to be applied to the rated performance data (RPD) as determined by testing and described in the following section. These factors are sensitive to the polymer composition of the rubber compound, which can be widely varied to produce a range of fender properties (Kumar 2014). The compound modulus (rubber stiffness) is a determining factor in fender performance and is greatly affected by carbon black dispersion. Rubber properties can be determined by thermogravimetric analysis testing (Trelleborg n.d.).

Specifications for rubber fenders should include the required energy, reaction, and sufficient definition of the rubber compound to be ensured of satisfactory life expectancy under most normal conditions (more than 15 years). The rubber compound should be specified as in compliance with the following line callout per ASTM D2000 (2001):

$$3BA x 20\ A14, C12, F17$$

where x may be either 4, 5, 6, or 7, as necessary to provide the specified performance. If vessels are berthing at temperatures below approximately 14°F (−10°C), the F17 callout should be changed to F19.

Most of the tests stipulated by the above line callout have no specific, required results, and some also cover various alternative test variations and lengths. Furthermore, in rubber testing, heat is used to increase the severity of a test. Specifying a test without also specifying a test temperature and a duration, in many cases, makes the specification worthless. The recommended line callout specifies durometer hardness, elongations, ultimate strengths, ozone concentration, test temperatures, and durations and sets the minimum acceptable values for each tested parameter. Other properties are often specified, but the value of doing so is somewhat questionable because it is not clear that any other properties have a well-established relationship to fender life. ASTM D2000 (2001) defines the parameters that directly affect longevity; it is a shorthand method to list all the necessary information in a succinct but complete manner. The much more impressive looking listing of all the separate tests, if all the necessary information were presented in the correct manner, would take more than a page. ASTM D2000 references 19 separate ASTM test standards, including multiple variations of many of them, and provides a code for stipulating 24 different test temperatures and the recommended minimum test results. Furthermore, it only recommends tests and results in certain compatible combinations, depending on rubber type and intended use. Certified compliance with the above ASTM D2000 line callout is all that is required to get satisfactory life out of a rubber fender in almost all applications. PIANC (2002) also specifies rubber physical properties for resistance to heat aging and ozone in particular, and it in turn references appropriate ASTM, ISO, and JIS test methods.

Performance Requirements and Testing

Performance testing is conducted to determine rated performance data (RPD) (i.e, rated energy capacity at given deflection and associated reaction force). Performance testing is carried out under ASTM F2192 (2002a) or PIANC (2002) requirements, which requires type approval and verification testing. Test results include the effects of rate of load application, velocity factor (VF), effect of temperature (TF), effect of angular compression (0° to 20°) and durability with a minimum of 3,000 cycles to rated deflection without failure. Pass/fail criteria allow a ±10% RPD performance tolerance for most molded shapes but up to ±20% for extruded shapes, and other tolerances may apply for other fender types. These tolerances apply only to the RPD and should be considered in final fender selection or pier design because a given fender unit could have 10% less energy at a 10% higher reaction force.

Fender Panels and Facing Materials

Fender face panels are normally of a mild carbon or low-alloy structural steel that has a minimum thickness of around 1/2 in. where it is exposed on both sides and around 3/8 in. where it is enclosed or fully protected on one side. Panels must be sized for the allowable hull pressure and must be able to support the fender reaction force with an appropriate factor of safety. Panel face edges should be beveled to reduce that chance of vessels and/or lines hanging up. The structural steel panel face is normally covered with a low-friction facing material, such as described in the following text, that may be installed in one large pad or any number of smaller pads to facilitate replacement.

Despite its seeming similarity to other polyethylenes (PEs), such as high-density (HDPE) and high-molecular-weight (HWPE) polyethylenes, ultrahigh-molecular-weight (UHMW) polyethylene is vastly superior in performance to these other, commonly supplied materials. So-called synthetic resin is usually HDPE. In typical fender applications, UHMW outlasts other materials, especially HDPE, by at least five times. Since it is chemically similar to all other PEs, most typical material specifications are useless for separating it from the less-durable forms of PE.

Currently, there is only one standardized test that distinguishes UHMW from all other materials: ASTM D256, Method B, the Izod impact test (ASTM 2002b). UHMW is the only known material that does not fail when subjected to this test. Thus, there is no acceptance value. The acceptance value is "no fail." Some engineers have taken to the practice of specifying a nonofficial test called a "double-notch" Izod impact test. They like this test because UHMW actually fails it, and they can then specify numbers to set acceptance thresholds. Higher values do not necessarily provide superior performance, and sometimes slightly the opposite, but the resulting materials are more expensive to make. For marine applications, "no fail" of the standard Izod test is the preferred acceptance value. This result provides arguably the best performance and the lowest cost.

There are several proponents of specifying cross-linked and/or virgin UHMW. However, it has not been shown that there is any benefit to either of these added requirements, other than to increase the profit to the vendor. Cross-linking has shown benefits in other types of applications, but for marine applications, the added rigidity this method produces is not necessarily beneficial. Often, it is slightly detrimental. Using virgin material, rather than recycled or blended materials, does have some useful benefits, but in most cases they may not be worth the added cost. There is now becoming available a new kind of processing, cryogenic grinding, for the recycled material that seems to give it properties virtually identical to virgin material for fender applications. The use of blends of virgin and cryogenically ground, reclaimed UHMW provides probably the ultimate cost/benefit ratio in most cases.

In all cases, UHMW should be UV-stabilized by the use of at least 2.5% stabilizer, and if colors other than black are specified, the dyes should be UV-stabilized as well. These precautions cost almost nothing, but they may not be incorporated if they are not specified, and they increase useful life four- to sevenfold.

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CHAPTER 6

Mooring Loads and Design Principles

Mooring loads often govern the required lateral load capacity of a pier or berth structure. They are induced by the action of wind, current, and waves on the moored vessel and are transmitted to the structure via direct breasting or through the pull of mooring lines. Within protected harbors, wind and current loads usually are estimated for some assumed limiting wind and current velocities, and the effects of waves and harbor motions are neglected. Today with marine terminals in more exposed locations, wave loads cannot be neglected and, in fact, may govern design. With larger vessels transiting more crowded harbors, the effects of passing vessels on vessels already moored must be considered. Even in protected harbors, vessel movements in response to long-period waves and harbor oscillations known as seiches may cause damage to vessel and pier alike, even when winds and currents are calm. Mooring breakaway incidents have demonstrated the importance of conducting a mooring analysis and adhering to sound mooring practice.

This chapter introduces some fundamental concepts of moored vessel behavior and the layout of mooring lines and hardware for vessels moored at fixed structures. Evaluations of wind, current, wave, and other load sources are treated separately. Because of the complexities in the determination of wave loads and other dynamic effects, mathematical and physical scale models are often used, and this subject is introduced at the end of the chapter. This chapter is oriented to vessels moored at fixed structures; however, most of the methods of calculating mooring forces are common to freely moored vessels as well. The subject of freely moored vessels at single point and spread moorings as well as small-craft moorings are treated further in Section 9.7.

6.1 Mooring Principles

Mooring forces acting on a ship at berth arise from the following sources:

- Wind,
- Current,
- Passing vessel effects,

- Hydrodynamic standoff forces,
- Wave action,
- Seiche and long-period wave effects,
- Tidal variations and vessel draft changes,
- Operational loads, and
- Ice.

In addition to staying secured in its berth, a moored vessel must remain as motionless as possible within a limited range of acceptable motions. At most harbor and nearshore locations, it is usually sufficient to design the mooring system components for the maximum expected or prescribed limiting wind plus current loads. Mooring systems adequately designed for appropriate combined wind and current loads usually possess sufficient capacity to cope with minor wave and water level changes, wind gust effects, and so on. Along riverbanks with strong currents or narrow channels, however, significant off-berth standoff forces may arise because of locally increased water velocities and resulting negative pressures caused by passing vessels or strong currents. At offshore berths and exposed nearshore sites, wave action also must be considered. Certain harbors are affected by long-period wave action, such as surging motions, sometimes referred to as range action or *seiching*, which can result in large mooring line forces. In areas of large tidal range, especially when combined with rapid cargo-transfer operations, mooring lines may be subject to rapidly changing lengths and angles, resulting in increased line loads. Large vertical forces may result from friction between the vessel and the fender facing. Warping and shifting of the vessel within its berth may impose longitudinal rubbing forces. Ice is not usually a factor in mooring design loads, but it may need to be evaluated in cold regions, and especially in areas subject to moving ice floes and for operational considerations as discussed in Section 6.11. The importance of understanding mooring principles and practice cannot be overemphasized because there have been many serious mooring incidents, including breakaways, and some that have resulted in loss of life (COPRI 2014). Mooring incidents are investigated and reported by the U.S. Coast Guard, Marine Safety Center (MSC), and the National Transportation Safety Board (NTSB) (see Appendix 3 for web addresses).

The general approach to calculating mooring loads on piers, wharves, and berthing structures depends somewhat upon the vessel size and type, site exposure and environmental conditions, and amount of information available describing those factors. Vessels are normally secured to piers and wharves by lines consisting of wire or synthetic rope temporarily secured to mooring hardware such as bollards, bitts, and cleats; a simple traditional approach is to provide mooring hardware based upon experience in lieu of determining loads from specific vessels. For example, for smaller vessels and for general and bulk cargo ships up to around 20,000 tons displacement at protected harbor locations, it may be adequate to provide mooring bollards or bitts of specified capacity at suitable locations, usually between 30- to 60-ft

Table 6-1. Mooring-Point (Bollard) Loads for General and Bulk Cargo Ships

Vessel Displacement (tons)	Bollard Load (kN)[a]
≤2,000	100
10,000	300
20,000	600
50,000	800
100,000	1,000
200,000	1,500
>200,000	2,000

Note: 1 kN = 224.8 lb; 1 m = 3.28 ft.
[a]For exceptional wind, current, or other adverse effects, increase the above values by 25%. Per EAU (2004), provide corner bollards of 2,500-kN capacity for vessels >100,000 tons. Assume bollard spacing of 15 m to 30 m.
Source: Data from BSI and EAU.

centers along the wharf face. A similar approach may be taken for even larger vessels when little is known about the specific vessels to be berthed. The required capacity of the mooring hardware can be presumed from some guideline standard such as BSI (2013, 2014), EAU (2004), and OCADI (2009), or other accepted practice. British (BSI 2013) and German (EAU 2004) code recommendations for bollard capacities are summarized in Table 6-1. Another approach is to size the mooring hardware to exceed the breaking strength of the vessel's docking lines, if this information can be reliably obtained.

The mooring point loads are distributed into the overall pier structure, as described in Section 7.3. Equivalent distributed mooring loads for oceangoing vessels usually range from 1,000 to 2,500 lb/ft of pier face applied horizontally at deck level for vessels from 2,000 to 20,000 tons displacement (DT), to more than 3,000 lb/ft for the largest vessels. Longitudinal mooring forces are usually on the order of 25% to 50% or less of the lateral forces. Vertical uplift on bollards also must be considered.

In general, for vessels around and exceeding 20,000 DT, for special vessels such as those with high superstructures, and/or where severe site conditions exist, a mooring analysis should be conducted as described in the following section. The maximum probable loads should be calculated for each mooring point for the entire range of vessel sizes and types expected. This step involves calculating the combined forces and moments produced by the design-level wind plus current plus equivalent static wave force and/or other forces, as applicable, and with regard to the elasticity and geometry of the mooring lines. The calculations may be simplified in some instances by assuming that only bow/stern and/or breast lines take the lateral loads, and spring lines take the longitudinal loads. Refer to Fig. 6-1 for mooring line and layout definitions.

Another simplistic approach is to assume an unequal distribution of the total mooring force among the mooring points. According to a simple rule of thumb, for

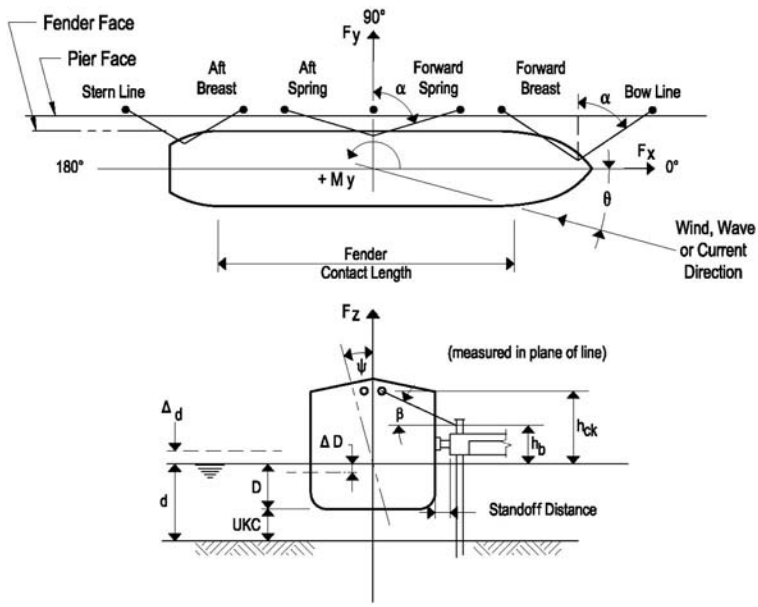

Fig. 6-1. Vessel mooring lines and berth layout definitions

example, when the vessel is moored at six points, one-third of the total force should be assumed at any one point, and when the vessel is moored at four points, one-half of the total force should be assumed to act at any single point. Calculation of line loads is discussed at the end of this section. In cases where wave loadings may be severe or long-period waves or range action are present, a more rigorous analytical treatment and/or the use of physical models may be required, as described in Sections 6.9 and 6.10.

A freely floating vessel possesses six degrees of freedom of motion—three translational and three rotational, as illustrated in Fig. 6-2. These possible motions are defined as follows with respect to a Cartesian coordinate system centered at the vessel's center of gravity (c.g.): surge, sway, and heave represent longitudinal, transverse, and vertical displacements in the x, y, and z directions, respectively; roll, pitch, and yaw represent rotations about the x, y, and z axis, designated by ψ, ϕ, and θ, respectively. Heave, roll, and pitch motions are acted upon by the restoring force of gravity; so a free-floating vessel possesses natural periods of roll, heave, and pitch, as described in Section 9.3. The periods of heave and pitch are usually on the order of one-half to two-thirds of the roll period. In a seaway, the various motions may interact via exchange of energy between modes, resulting in even more complex, coupled motions. This is especially true for coupled heave and pitch and for surge and yaw. Surge, sway, and yaw motions only exhibit a natural period when acted upon by some restoring force, such as mooring lines and fenders, and it is these motions that are of principal interest herein. The dynamic behavior of the ship and mooring system is addressed in Sections 6.8 and 6.9.

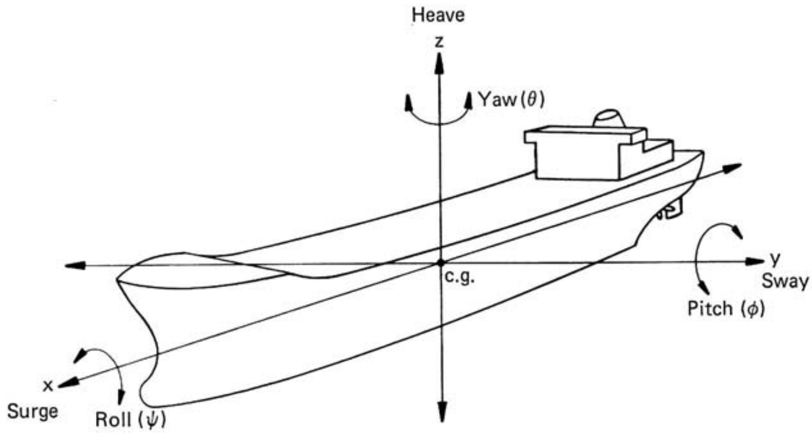

Fig. 6-2. Vessel motions definition

Environmental disturbing forces manifest themselves in a spectrum of periodic forces covering a wide range of periods, as introduced in Section 3.4. A moored ship may respond selectively even to subtle influences, such as the low-frequency components of the wave drift force discussed in Section 6.8. When the ratio of the ship/mooring system natural period (T_n) almost coincides with the period of the disturbing force (T_f), then resonance results in a dynamic amplification that theoretically tends to infinity without damping. Typical natural periods of moored ships range from around 20 s for vessels of 3,000 DT to 60 s and longer for vessels of 100,000 DT and larger.

Mooring Arrangements

Fig. 6-1 illustrates the general arrangement and nomenclature for a vessel moored alongside a pier or quay, breasting against a fender system and being restrained by mooring lines. Bow lines, sometimes referred to as head lines, and stern lines are attached to the respective ends of the vessel and usually make an angle of up to around 45° with the face of the pier (larger angles are not recommended), so that they provide some degree of both lateral and longitudinal restraint. Breast lines are almost normal to the pier face and provide only lateral restraint, whereas spring lines run fore and aft, usually at an angle of 5° to 10° to the pier face, and provide only longitudinal restraint. The total length of a mooring line, or hawser, from the connection point on the pier to the connection point on the vessel, is denoted by l. As a practical matter, mooring lines should not be less than 100 ft long, to allow for change in tide and ballast conditions and to provide sufficient elasticity. The horizontal angle with a line normal to the vessel is denoted by α, and the vertical angle in the plane of the hawser by β. Vertical angles should not exceed 25° to 30° maximum and vary with the stage of tide and the vessel's ballast condition. In general, a minimum of four lines is required to safely moor a vessel, and a dozen or more lines may be required to secure larger vessels. Mooring arrangements should in general be as symmetrical about the vessel's centerline as possible, and the number of lines attached to a given mooring fitting should be minimized. Bow, stern, and breast lines should not be allowed to cross one another. Further description of typical pierside mooring line layouts can be found in the ASCE/COPRI, Manual of Practice No. 129, *Mooring of Ships to Piers and Wharves* (ASCE/COPRI 2014).

Tankers, gas carriers, and many larger bulk carriers typically berth at dolphin or "sea island"-type berths. A tanker-berth type of mooring arrangement is illustrated in Fig. 6-3. The vessel is breasted against two or more breasting dolphins, with mooring lines running to smaller mooring dolphins, usually interconnected by catwalks for personnel access. The breasting dolphins should ideally be spaced approximately one third of the design vessel's length overall (LOA) symmetrically about the center manifold. The minimum to maximum spacing of 25% to 40% LOA allows for some accommodation of a range of vessel sizes and for maintaining fender contact within the vessel's parallel midbody length. Bow, stern, and breast lines should preferably have a maximum angle α of 15° and spring lines a minimum α of 80°, in accordance with the Oil Companies International Marine Forum *Mooring Equipment Guidelines,* 3rd edition (OCIMF 2008). In general, this type of berth should be laid out symmetrically. Note that because the dolphins are set back from the berth line,

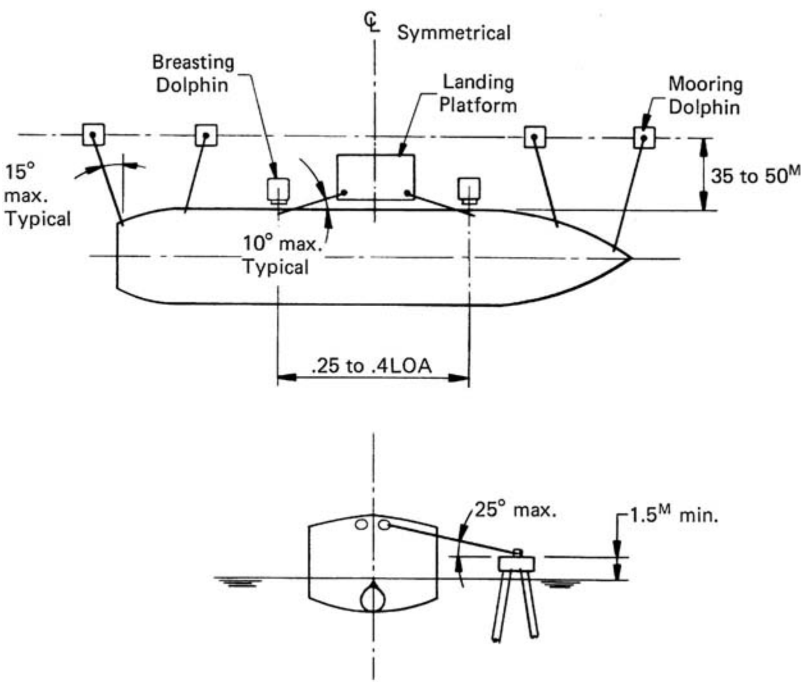

Fig. 6-3. Tanker berth mooring line layout

Source: Adapted from OCIMF (1997)

the bow and stern lines are more nearly normal to the vessel's hull than they otherwise would be, allowing for greater lateral restraint. A vessel may be safely moored within its own length, provided that the dolphins and hardware are properly designed. Bulk cargo terminals also may use this concept, frequently using a greater number of dolphins, especially in cases where the ship needs to be shifted in position for access to all its hatches by loading/unloading equipment. Specific guidelines for tanker berths and recommended mooring practice can be found in PIANC (2012a), MOTEMS (2011), and OCIMF (2008).

The overall adequacy of a vessel's mooring arrangement is determined not only by its structural integrity and safety under storm conditions, but also by the limitation of vessel movements at berth to prevent possible damage to the ship, pier, and loading equipment. Allowable vessel movements are covered in Section 6.11, and operational limits caused by wind are covered in Section 3.4. Special care must be exercised in the location and orientation of a berth to avoid exposure to potentially damaging waves and/or nodal points in harbors subject to seiche or ranging action (see Section 3.4). In the planning of a facility in an exposed location, a statistical approach is often taken, in which the percent frequency of time the berth is occupied and subject to conditions exceeding certain prescribed limits (e.g., limiting wave height by direction) can be compared with the economics of vessel turnaround time.

Fig. 6-4 shows a variable-orientation berth for an exposed offshore bulk terminal. The arrangement of dolphins and the ship loader characteristics allow for a choice of six orientations, increasing the total berth availability to 90% as opposed to 70% for the most favorable single orientation. This facility and other multiple-orientation berth arrangements have been described by Sugin (1983) and by Soros and Koman

Fig. 6-4. Variable orientation. Offshore loading terminal at Punta Colorada, Argentina

Source: Photo courtesy of Soros Associates, Consulting Engineers, New York

(1982). In general, berths should be oriented as much as possible with the prevailing current and/or wave direction in order to minimize current forces and increase controllability during berthing.

Although ships are usually moored via deployable flexible lines, permanently moored vessels, such as floating dry docks, habitats, museums, and restaurants, may be rigidly moored by mechanical means such as guide piles or spuds with articulated grippers (see Fig. 7-25) or by other means, as described in Section 9.7. Other "shore-based" approaches to mooring alongside a pier or wharf include a novel system known as MoorMaster units by Cavotec, which is an automated mooring system consisting of vacuum pads in articulated frames that grip the sides of the ship, forming a semi-rigid connection that allows for some degree of vessel motion while holding the ship securely alongside. Because such devices are proprietary, the manufacturer must be consulted for specific information. De Bont (2010) reports on a dynamic analysis of a containership moored by these units under wave action. The ShoreTension system consists of pier-mounted hydraulic units that maintain constant tension in the mooring lines throughout tide and draft changes and changing environmental conditions (van der Berg 2011). Certain fast-turn-around ferry and transfer operations use shore-mounted winches with lines that are passed to the ship from the shoreside and are secured to dedicated mooring hardware at fixed locations on the ship. Such systems are also proprietary and are described in further detail by Clark (2009).

Mooring Design Criteria

It is generally considered unnecessary to design a mooring facility for the most extreme (e.g., 100-year) storm condition with the maximum vessel alongside. Hunley and Lemley (1980) point out that, based upon available studies, 90% of the winds experienced at commercial ports are below 35 knots, discounting gusts of less than 5-min duration. Standard practice in many instances is to assume that vessels put to sea in extreme weather conditions when winds exceed certain threshold values, often taken to be around 50 to 60 knots. Where such assumptions are not considered valid, some prescribed recurrence level of extreme wind conditions, such as a maximum annual or 50-year return period event, has been adopted by the U.S. Navy in *Design: Moorings* (DOD 2005a). For permanently moored vessels, such as floating dry docks, floating piers or storage vessels, and mothballed vessels, the maximum winds associated with 100-year return periods should be used. Conversely, for temporary facilities with short design lives and/or restricted vessel visits, the extreme winds associated with an annual or 10- to 25-year return period may be applied at the designer's discretion. At terminals serving a mix of vessel sizes, and especially for those with rapid vessel turnaround times, the joint probabilities of the largest vessel being alongside during the occurrence of 50- to 100-year storm conditions for the site are generally acceptably low. For most active marine terminals relatively protected from wave action, a minimum design wind speed of around

60 knots from any direction combined with the maximum spring tidal current acting on the largest, most frequently calling vessel seems to be a reasonable minimum design condition in the absence of other information. Design wind speeds must be referenced to specified durations and elevations, as described in Section 6.5. Static wind and current analysis may be applied to vessels moored at "protected" locations.

DOD (2005a) requires that special consideration (beyond static wind and current analysis) be given when the following conditions pertain:

- Wave heights exceed 1.5 ft for small craft or 4 ft for larger vessels.
- Winds exceed 45 mph for small craft and 75 mph for larger vessels.
- Currents exceed 3 knots.
- The site is exposed to long waves, seiche, or tsunamis and/or passing vessel/ship wave effects.
- The site is exposed to hurricanes or typhoons, ice conditions, and other specific natural hazards.

Dynamic analysis is required for the above wave, seiche, and/or hurricane conditions and for single-point moorings (SPMs) in general. The wave period may be just as important a criterion as wave height for determining its potential effect on moored vessels. In particular, the peak spectral period (T_P, the period at which most of the sea's energy is concentrated) is most critical. For oceangoing vessels, when T_P exceeds approximately 4 s, waves begin to have a significant effect, depending upon the vessel's orientation to the sea. Wave conditions are described further in Section 6.8. The U.S. Navy currently designs for four basic mooring service types per DOD (2005a), as follows:

- Type I—Mild weather mooring; winds up to 34 knots, currents less than 1 knot
- Type II—Storm mooring; winds up to 64 knots, currents less than 2 knots
- Type III—Heavy weather mooring; design for 50-year site-specific event
- Type IV—Permanent mooring; design for 100-year site-specific event

Navy ship hardware must be designed for a minimum service Type III condition (NAVFAC 2000). Seelig (2001a) provides an overview of U.S. Navy mooring practices. Additional requirements for heavy-weather moorings and specific U.S. Navy facilities are described by Seelig and Curfman (2000).

OCIMF (2008) recommends the following minimum design criteria for very large crude carrier (VLCC) vessels' mooring equipment where they are not exposed to wave action:

- A 60-knot wind (as measured at 10 m and corresponding to a 30-s gust duration) from any direction, plus one of the following:
    - 3-knot current from 0° and 180°

  - 2-knot current from 10° and 170°
  - 3/4-knot current from abeam
- All current forces based upon a $d/D = 1.1$ to almost 1.0. For combined loading, $d/D = 1.1$ to 3.0 must be investigated; for a vessel in full load to light condition, the current speed is average over the vessel's draft.

Under the above conditions, the load in the most heavily loaded line should not exceed 55% of the minimum breaking load (MBL) of the line for wire rope and less for nylon and synthetic lines, as described in Section 6.3. Mooring lines should be between 35 and 50 m long and preferably pretensioned to 10% MBL. Currents for the terminal facilities design should be site specific instead of those required for ship equipment given above.

The California State Building Code includes provisions for marine oil terminals (MOTs) originally developed by the California State Lands Commission (SLC) entitled *Marine Oil Terminals Engineering and Maintenance Standards* (MOTEMS 2011), which provides design criteria for both new and existing MOTs. MOTEMS (2011) requires that wind, current, waves, and combinations thereof shall be defined as limiting conditions at each berth with and without product transfer and that a statement of terminal operating limits be prepared. MOTEMS (2011) defines risk classifications where winds >50 knots and/or currents >1.5 knots are considered "high risk." Wind, current, and wave design criteria are summarized as follows:

- Wind: Design for maximum 25-year return period (Rt) for new MOTs and determine threshold criteria for vessels to vacate berth for existing MOTs. Wind velocity should be based on 30-s duration gust, and the designer must check a minimum of eight directions at 45-degree increments.
- Current: Site-specific current data are required for current velocity >1.5 knots based upon at least one year of record, and the designer must check two directions, flood and ebb, and two tide levels, min/max with min/max draft for worst combination with wind direction.
- Waves: If the significant wave period $T_s > 4$ s for the maximum annual significant wave height H_s, then a dynamic analysis is required. The designer must evaluate possible seiche and tsunami effects.

At offshore locations, design mooring conditions must be specified under which vessels of a given size would remain alongside. Such design criteria often are presented in terms of prescribed limiting conditions for berthing operations and terminal operations, and/or maximum conditions under which the vessel would remain alongside, as well as survival conditions for the terminal facilities without the ship in berth. For example, design criteria for the offshore berth depicted in Fig. 6-4 are described as follows, after Sugin (1983):

- Maximum ship size: 60,000 DWT
- Water depth at berth: 42 ft with 30-ft tide range
- For design berthing conditions:
  - Wind = 5 to 30 knots
  - Wave height = 4 to 6 ft (head seas)
  - Current = N.A.
- For design operating conditions:
  - Wind = 10 to 35 knots
  - Wave height = 6 to 8 ft (head seas)
  - Current = N.A.
- For survival condition without ship in berth:
  - Wind = 80 knots
  - Wave height = 12-ft significant wave (H_s)/29-ft maximum wave (H_{max})
  - Current = 2 knots

The above conditions must be analyzed over a range of tide levels and wind and sea directions to obtain the worst load combinations. Sugin (1983) also has summarized design criteria for several other offshore bulk loading terminals.

6.2 Mooring Analysis

Once the berth layout and design mooring conditions have been determined, then wind, current, and equivalent static wave and other forces, as applicable, can be calculated, as described in the following sections, and combined to obtain the maximum total longitudinal and lateral force components and yaw moment. The calculations may need to be carried out over a range of wind directions and water depths, but only for the combined maximum conditions that are likely to occur simultaneously. Because wind forces are higher with a vessel in light/ballast condition and current forces are higher for vessels at full load, deep draft condition, a range of combined wind and current scenarios needs to be considered. When dynamic wave loads are involved, they cannot, in general, be directly combined with steady wind and current loads by linear superposition; such calculations must be regarded as approximate and subject to dynamic analysis, as described in Sections 6.8 and 6.9. Passing vessels, hydrodynamic standoff forces, and long-period waves may produce large mooring loads, but in general, these conditions do not always act simultaneously with the maximum design wind, wave, and current conditions.

For a given site, berth orientation, and mooring arrangement, the design process generally proceeds as follows:

1. Define environmental conditions.
2. Calculate environmental loadings.

3. Calculate mooring system response and component loads.
4. Determine factors of safety.

Steps 2 through 4 may have to be reiterated for alternative berth orientations, and Steps 3 and 4 for alternative mooring arrangements. Factors of safety on mooring system components vary widely with specifying authority and with line and equipment type and are addressed in Sections 6.3 and 6.4. The designer should of course also consider the severity and statistical probability of design environmental conditions, the level of confidence in design assumptions and calculation methods, and the nature and consequences of a failure. Redundancy should be provided as much as is practical.

Individual mooring line loads are calculated by balancing the sum of the static wind plus current longitudinal (F_x) and lateral (F_y) force components and yaw moment (M_y) about the vessel's geometric center and considering the geometry and elasticity of the mooring lines.

As an aid to determining the component line loads and identifying critical line loads, it may be useful to plot the line loads and total elastic restoring force (F_R) versus vessel surge and sway excursions ($\varepsilon_x, \varepsilon_y$), as illustrated in Fig. 6-5. The mooring line force is then a function of its stretched length, assuming some initial at-rest length. Mooring lines are normally pretensioned to remove the initial sag and sometimes to precompress the fenders at exposed sites and to help maintain the ship's position. The effect of any pretension in the line also should be accounted for. Normally a pretension of approximately 10% MBL removes the initial sag from a synthetic line. The F_R then can be determined and balanced against the applied forces. Manual calculations may be simplified by neglecting the effect of spring lines on lateral restraint and of breast lines on longitudinal restraint. Breast lines for lateral restraint should be grouped as much as possible at the bow and stern for efficiency as well as to simplify analysis. In order to facilitate the analysis, then the total lateral force (F_y) and yaw moment (M_{xy}) can be converted to equivalent lateral forces at the bow, forward perpendicular (F_{fp}), and stern, aft perpendicular (F_{ap}), as follows:

$$F_{fp/ap} = \frac{F_y}{2} \pm \frac{M_{xy}}{\text{LBP}} \tag{6-1}$$

where LBP = length between perpendiculars.

In some cases, wind and current force data may already be provided in this format. The $\pm$ in the above equation relates to the direction of M_{xy} in relation to F_{fp} and F_{ap}. It is most important, therefore, that the sign convention of the reported force and moment data be carefully observed because there is no universally accepted convention for wind and current data. Individual line loads should be tabulated and compared with the line's allowable load so that critical, highly loaded lines can be rearranged or resized.

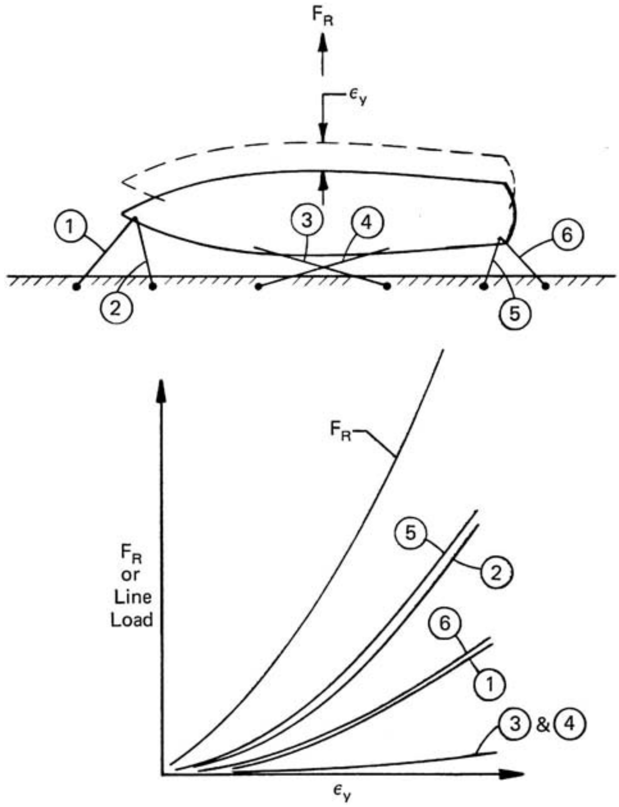

Fig. 6-5. Example plot of mooring line stiffness

The OCIMF (1997) guidelines, predecessor to OCIMF (2008), for tanker mooring equipment provides instructions for manually calculating line loads and identifying the most critically loaded lines. In order to calculate mooring line loads with reasonable accuracy and thus provide a high degree of confidence in the security of a mooring layout, the OCIMF guidelines emphasize the following good mooring principles:

- Lateral and longitudinal restraint functions are separated.
- Layouts should be as symmetrical as possible.
- Vertical line angles (β) should be less than 25°.
- Bow and stern line deviations from true breast lines (α) should be less than 45°.
- All lines should be of similar material, construction, makeup, and size.
- Lateral lines should be effectively grouped at the bow and the stern.

The tension in lateral restraint developed in any line is proportional to cos α/l, and the component of tension effective in resisting the lateral load is proportional to

$\cos^2\alpha/l$. The maximum restraint capacity of a mooring layout is limited by the allowable or ultimate tension capacity of the most heavily loaded line. The most critical line in a group is the one with the highest value of $\cos\alpha/l$ or $\cos\alpha\cos\beta/l$, so the efficiency of any line in resisting an imposed load is related to the geometry of the critical line as well as its own. The total lateral restraint (R_y) can be found by grouping lines at the bow and stern generally at the forward and aft perpendiculars (f_p and a_p) and summing the effective line tensions divided by the lateral restraint of the most efficiently placed and hence critical line (denoted by subscript c) with the highest value of $\cos\alpha_c/l_c$ times $\cos\beta_c$ times the allowable line load=MBL divided by a factor of safety (FS), assuming that all lines are of the same size and construction. The length (l) of line includes the length from the point of attachment to the shore bollard to the vessel's chock plus the length across the vessel's deck from the chock to its point of attachment on the ship's mooring hardware or winch. Where wire rope lines are fitted with synthetic tails, the length (l) should be increased by considering a length of tail as an equivalent wire with length greater than that of the tail in proportion to the ratio of percent elongations of the tail and wire at the same load. It is also assumed in the following simplified analysis that the line's elastic stretch properties remain essentially linear up to 0.55 MBL. Factors of safety are discussed in Section 6.3. The lateral restraint capacity at each end of the vessel based on the first line to fail is then given by

$$R_{y(fp/ap)} = \frac{\text{MBL}}{\text{FS}}\left(\sum\frac{\cos^2\alpha\cos^2\beta}{l}\right)\left(\frac{l_c}{\cos\alpha_c\ \cos\beta_c}\right) \tag{6-2}$$

For symmetrical line layouts with $\beta \le 25°$, Eq. (6-2) can be simplified to

$$R_{y(fp/ap)} = \frac{\text{MBL}}{\text{FS}}\left(\sum\frac{\cos^2\alpha}{l}\right)\left(\frac{l_c}{\cos\alpha_c}\right)\times\ \cos\beta_c \tag{6-3}$$

The nominal capacity is the sum of the allowable line load times the number of lines (N), and hence the efficiency of the layout is given by

$$\text{Eff.} = \frac{\text{MBL}}{\text{FS}}\times\frac{N}{R_y} \tag{6-4}$$

The longitudinal capacity can be calculated in a similar manner and simplified if it can be reasonably assumed that all spring lines are of almost the same length with similar α and β; hence, the total longitudinal restraint (R_x) in either direction, forward or aft, is given by

$$R_x = \frac{\text{MBL}}{\text{FS}}\times N\times\sin\alpha_c\ \cos\beta_c \tag{6-5}$$

The previous equations can be organized in tabular form, and thus this methodology lends itself to spreadsheet applications. The U.S. Navy design data sheet (NAVSEA 1987) provides step-by-step instructions for calculating mooring line loads that are readily amenable to spreadsheet solution. This document has been appended to Part 5 of the American Bureau of Shipping (ABS) naval vessel rules (ABS 2004), which is a limited-distribution document but should be made available to those involved with relevant U.S. Navy projects. Simplified manual calculation procedures are also presented in NAVFAC (1986a). The spreadsheet program EMOOR, developed by the U.S. Navy as a preliminary design and planning tool for evaluating ship moorings at piers and wharves, is presented by Seelig (1998) and provides a simplified method for the preliminary evaluation of mooring line loads and the efficiency of the mooring arrangement.

The computer program OPTIMOOR calculates wind, current, wave, and other mooring forces, allows for water level and draft changes, determines mooring line and component bollard loads, and includes OCIMF coefficients in its database (Flory and Ractliffe 2012). The NAVFAC program FIXMOOR (NAVFAC 2008) has been developed into an interactive web-based program that predicts static wind and current loads and mooring line loads for ships moored alongside and includes access to the U.S. Navy ships database. The program is available through the Whole Building Design Guide (WBDG) website (see Appendix 3), but it requires registration and a password from the Navy. Chernjawski (1980) presents an analytical method for a computer application for determining mooring line forces in response to wind, current, and equivalent static wave loads, which also considers the hydrostatic restoring forces in response to vessel trim, list, and immersion, and the nonlinear modulus of elasticity of the mooring lines.

The methods presented here are suitable for static mooring analysis, assuming steady-state wind and current and linear behavior of mooring lines and fenders. Dynamic analysis considers the effects of unsteady wind and current, wave action, and other dynamic effects, such as passing vessels and suddenly applied loads and the resulting vessel motions and mooring system response. A simple example of dynamic behavior is the response of a moored vessel to a sudden strong wind gust of sufficient duration to mobilize the vessel. A ship moored alongside being pushed away from a pier moves outward until the increased force is balanced by the increased mooring line tension, at which point it begins to decelerate but continues to move away from the pier because of inertia, resulting in increased stretch and corresponding increased line tension. Once the ship's motion has been checked, the elastic energy stored in the lines pulls the ship back against the fenders and may result in repeated cycles of bouncing off of the compressed fenders until checked by damping due primarily to hydrodynamic resistance. If the lines were initially slack, peak "snatch" loads as the lines suddenly came up taut could be as much as twice the static load associated with the wind gust speed (Clark 2009). Precompression of the fenders would help alleviate such dynamic effects, and therefore it is advisable to pre-tension mooring lines to hold a vessel snug against the fenders at exposed locations. Under certain conditions, dynamic effects can result in mooring line and fender loads that greatly exceed the loads determined by static methods. When dynamic analysis is required (see Section 6.1), such dynamic effects are most accurately predicted by time history analysis, which requires relatively sophisticated modeling techniques and computer programs, as described in Section 6.9. The dynamic version of the program OPTIMOOR can also be used for various types of dynamic analysis and uses a response spectrum type analysis for wave loads, as introduced in Section 6.8.

The general design of marine structures to accommodate vessel mooring loads is addressed in the general port and harbor engineering literature cited in Chapter 1. ASCE/COPRI (2014) provides a concise overview of the mooring of ships at piers and wharves, and in addition to DOD (2005a) and OCIMF (2008), general mooring practice and design guidance can be found in the British (BSI 2013, 2014), the German (EAU 2004), the Spanish (ROM 1990), and the Japanese (OCADI 2009) standards. Study of design case histories can be most informative, as can be found among the various technical papers of the conference proceedings listed in Appendix 2. Bruun (1984) summarizes the contents of several important papers covering rational principles in mooring and fendering design presented at the Eighth International Harbour Congress that remain relevant today. Criteria and methods for calculating environmental forces, such as wind and current on moored vessels, are presented in DOD (2005a), OCIMF (2008), BSI (2013), and ROM (1990). An appendix to PIANC (2012a) provides a concise overview and comparison of the last of these three standards. Forces on moored vessels caused by wind, currents, passing vessels, wave action, and operational sources are addressed individually in the following sections of this chapter.

6.3 Mooring Lines

Mooring lines provide the critical link between the vessel and the berth structure, although the importance of their proper application to mooring safety often is overlooked. Mooring lines usually consist of synthetic fiber rope and/or steel wire rope. In general maritime usage, a *rope* becomes a *line* when put to some specific use, and in turn mooring lines are often referred to as *hawsers*.

Fiber Ropes

Fiber ropes may be constructed of three (plain-lay) or four (shroud-lay) bundles of wound fibers called *strands*, which are twisted together; alternatively, plaited or braided rope construction may be used. Plaited rope is made up of pairs of strands twisted in opposite directions, thus eliminating the problem of kinking resembling a knot in twisted ropes known as a *hockle*, and so are torque free. Eight-strand and

12-strand construction are most common for mooring lines on larger vessels. Braided rope construction is similar to plaited rope but is made up of many more, smaller yarns rather than strands. Double braid consists of an inner core of stranded rope with an outer cover of braided rope and is also torque free. Rope was traditionally made from natural fibers such as manila, sisal, jute, or hemp, but natural rope is uncommon today. Rope is most commonly made of synthetics such as nylon (polyamide), dacron (polyester), polypropylene, polyethylene, and sometimes a combination of these materials. More recently, high-strength/low-stretch materials, such as aramid ("Kevlar") and high-modulus polyethylene (HMPE), which has the equivalent strength of wire rope with much less weight (it floats), have been used on tankers (OCIMF 2002) and have been adopted as the standard by the U.S. Navy. HMPE ("Dyneema") has an elongation at break of from 2.7% to 3.5%, depending upon rope type. In addition to high strength, HMPE line has excellent resistance to shock loads, deterioration, UV light, and aging.

The minimum bending radii for all fiber lines should not be less than four rope diameters. The elastic stretch at 50% of break load ranges from around 9% elongation for broken-in polyester and polypropylene to around 12% or more elongation for polyamide. The tensile strength of a line can be characterized in terms of its minimum breaking load (MBL) based upon full-scale test results. Manufacturers' published MBLs may sometimes be based upon the mean breaking strength from a number of test results, so care should be taken in selecting line capacities and in comparing published strength values. The strength of a line of a given material and construction is proportional to the square of the diameter times a constant. Fig. 6-6 illustrates approximate load/elongation characteristics for selected mooring lines.

The elastic properties of a line are of particular interest here. A fiber line initially loaded to around 50% or more MBL does not return to its original length when the load is released. Under repeated loading to a given stress level, the line develops a stable hysteresis loop with relatively predictable elastic properties. Unfortunately, a modulus of elasticity for fiber lines cannot be accurately defined, and the designer must refer to the manufacturer's test data when such information is important. In general, elastic properties of "broken-in" lines should be used in most mooring analyses. Nylon line, in general, exhibits stiffer behavior under cyclic loading, and greater elongation at a given load when wet than when dry. The dynamic load elongation behavior of nylon line is dependent upon the initial tension and its magnitude in relation to the line's ultimate strength. Ropes usually are terminated by splices or knots, or by belaying to bitts or other mooring hardware. An eye splice is about 95% efficient, whereas the use of knots results in around 50% to 70% efficiency, where efficiency refers to the strength of the knot or splice as a percent of the line strength. Mooring lines should be fitted with thimbles at eyes and chafe protection at eye ends and where they may make contact at sharp bends. Additional information on fiber rope can be found in the *Fiber Rope Technical Manual* of the Cordage Institute (CI 1997).

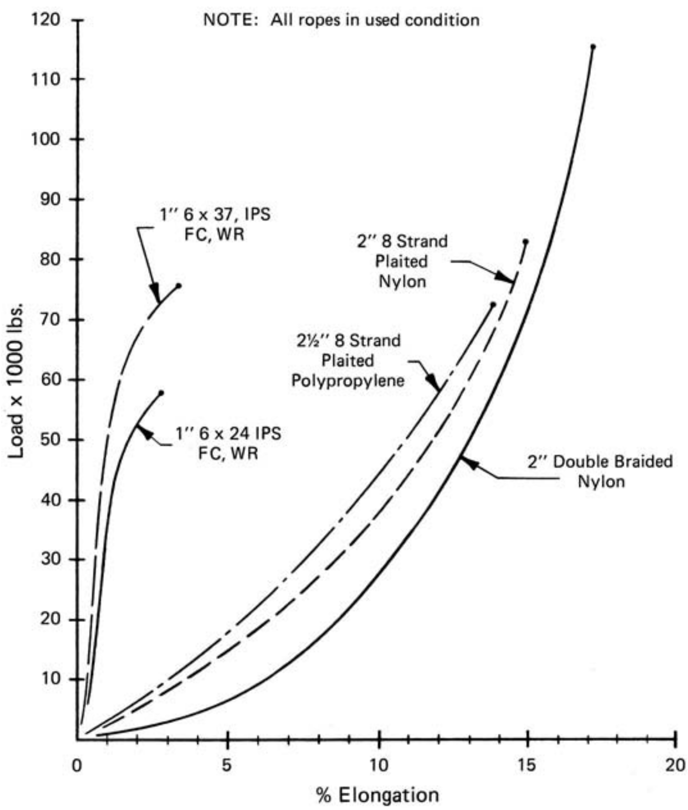

Fig. 6-6. Approximate load/elongation characteristics for selected mooring lines

Wire Rope

Wire rope is constructed of bundles of strands that are made up of individual wires twisted together. The rope construction is designated by the number of strands times the number of wires within each strand. Most wire rope used in mooring line applications consists of six strands; 6 × 24, 6 × 36, 6 × 37, and 6 × 41 constructions are most common for hawsers. The 6 × 37 classification, which consists of six strands with 27 to 49 wires per strand, is recommended by OCIMF (2008).Wire rope may be constructed of all solid strands or with a fiber rope core (FC) or independent wire rope core (IWRC) for greater flexibility. The standard practice of the wire rope industry is to use a factor of safety of five times the MBL for nominal wire rope working loads; however, OCIMF (2008) criteria allow for a factor of safety of 1.8 for tanker mooring lines. Wire ropes for mooring lines usually are made of preformed drawn wire strands of high tensile strength and should be galvanized for corrosion protection. The yield point of wire rope is not well defined but usually is on the order of two-thirds of the ultimate break strength. The elongation at break ranges from 2% to 5%. Approximate load elongation characteristics of a representative wire rope are shown in Fig. 6-6 for comparison with synthetic lines.

After repeated loading to around 30% of its ultimate tensile strength, wire rope becomes relatively elastic, and a modulus of elasticity (E_{wr}) can be defined within the elastic limit, which is typically around 55% MBL. Typical values of E_{wr} range from 10×10^6 to 22×10^6 psi, depending upon the rope construction and usage (Meals 1969). A value of around 12×10^6 psi is more representative for 6×37 IWRC with a solid metallic area approximately 51% of its nominal area. Wire-rope end connections may be made up by swaging on end fittings, splicing, or using wire-rope clips. Swaging and splicing are from 75% to 90% efficient, and the use of clips is from 75% to 80% efficient. The use of wire-rope clips is the most common means of connection because they can be readily applied in the field for rope of up to 1 5/8-in. diameter. A minimum of three clips, ranging up to six clips with increasing size, usually are required. Clips are spaced from 3 in. to 9 3/4 in. apart. Weight and strength characteristics and details of wire rope and synthetic rope construction and hardware can be found in the various manufacturers' product literature. General treatment of fiber and wire rope for mooring line applications can be found in NAVSEA (1999); in-depth treatment of wire rope is provided by the Wire Rope Technical Board (WRTB 2005).

Mooring Line Elasticity

In general, it is poor practice to mix wire and fiber lines as independent mooring lines because wire is much stiffer than fiber and carries most of the load. The total length of a line also affects its relative elongation. Wire rope and nylon, for example, can be effectively mixed in the makeup of individual lines, with the nylon used as a "tail" for ease of tying up and to take advantage of its elastic properties. The nylon also aids in absorbing shock loads. Typical nylon tail length for large ships such as tankers and gas carriers is 11 m, although longer lengths may be used at exposed locations with dynamic loads. The elastic stretch properties of steel wire with nylon tails must be treated as springs in series within their linear elastic range. The change in length (Δl) of a line under load is given by

$$\Delta l = \frac{T_1 l_0}{AE_{(T)}} \tag{6-6}$$

where

T_l = tension in line;
l_0 = initial unstretched length of line;
A = cross-sectional area of the line, wire, or fiber; and
$E_{(T)}$ = modulus of elasticity as a function of the tension.

Because fiber rope has no well-defined value for $E_{(T)}$, it should be determined from actual stress–strain curves for the specific use conditions. The elastic restoring force versus vessel excursion then can be determined from the geometry of the layout and elastic properties, as illustrated in Fig. 6-5. Wilson (1967) provides in-depth treatment of the elastic characteristics of mooring lines. Flory (1998) and Flory and Ractliffe (2005) discuss the effect of mooring line stiffness on the distribution of line loads and the importance of properly representing line stiffness in mooring analysis. Banfield and Flory (2009) discuss improved mooring line technology for tankers and gas carriers at exposed locations.

Mooring Line Factors of Safety

Factors of safety for mooring lines vary considerably between U.S. Navy and commercial practice (ASCE/COPRI 2014). According to DOD (2005a), an FS = 3.0 should be applied to the calculated line loads to determine the required MBL of wire rope and most synthetic lines. Polyamide (nylon) lines should have an FS = 3.5 to account for reduced strength when wet. OCIMF (2008), in contrast, requires the following FS be applied to the maximum calculated line loads to determine the required MBL (given as %MBL allowable):

- Steel wire rope: FS = 1.82 (0.55% MBL)
- Synthetics, except nylon: FS = 2.0 (0.50% MBL)
- Polyamide, nylon, wet: FS = 2.22 (0.45% MBL)
- Rope tails: for wire rope, FS = 2.28 for synthetic tails and 2.5 for nylon tails
- For synthetics: FS = 2.50 for synthetic tails and 2.75 for nylon tails
- Joining shackles: FS = 2.0 or > safe working load (SWL) of the line

Mooring Lines Carried by Ships

The size, type, and number of mooring lines carried by a vessel are normally determined in the ship design phase to safely moor the vessel with respect to environmental criteria, such as standard criteria, in accordance with OCIMF (2008) for tankers and gas carriers or the vessels' intended ports of call for other vessels. The material and construction of the mooring lines to be provided are of utmost importance.

Wire rope lines are generally preferred for large vessels, especially where limiting of vessel movement is critical and dynamic loads are low. The preferred construction is 6 × 37 class; 6 × 36 or 6 × 41 IWRC per OCIMF (2008). Wire rope sizes typically range from about 1 in. up to about 1 3/4 in., which is considered a practical upper limit for handling considerations, although larger diameters are sometimes used. For VLCCs, a minimum of 1 5/8-in. diameter is recommended. Minimum break loads thus range from around 40 to more than 200 kips. Wire rope mooring lines are normally fitted with fiber line tails, as previously described, for which nylon of eight-strand or double-braid construction is normally preferred, although polyester tails may be used, especially where durability is more important than stretch. Tails must be of higher strength than the wire ropes they are attached to. An increase in strength of 1.37 × MBL of the wire rope is recommended by OCIMF for wet nylon and of $1.25 \times$ MBL for polyester and other synthetics.

Polyester rope of eight-strand or double-braid construction is generally preferred for mooring lines on smaller vessels, where ease of handling, flexibility, and lower line loads are the more important considerations. Typical rope sizes range from around 1 1/2 in. up to about 3 1/8 in., which is considered a practical upper limit for handling, although larger lines are sometimes used. Minimum break strengths typically range from about 40 kips to more than 200 kips for 3-in. double braid. Eight-strand polyester rope is approximately 70% as strong as double braid for the same nominal diameter. Polypropylene lines are carried by many vessels, but it is not recommended by OCIMF because of its lower strength; low melting point, which can result in damage under dynamic loads; and low UV resistance, which often results in shorter service life. It is used by many smaller vessels, however, because of its low cost, light weight, and reasonable durability. Polypropylene also floats and may be a suitable choice for the "first lines ashore" carried by many vessels to facilitate berthing and initial tie-up but should not be considered part of the final mooring arrangement. Polypropylene may also be a good choice for the much smaller size "messenger" and pickup lines used to transfer the mooring lines between ship and shore.

HMPE rope is much stronger than steel relative to its weight and is replacing wire rope on many large vessels, such as tankers, and is becoming the standard for U.S. Navy vessels. HMPE ropes have similar strength and stretch characteristics to wire rope of the same diameter but are much lighter and float in water so that they are much easier to handle.

Vessels must also carry an adequate number of mooring lines to safely moor the ship at its intended destinations. For smaller ships, say, up to around 2,000 DT, six to eight lines should be provided, where six lines should be considered an absolute minimum. Larger vessels, such as VLCCs, may carry 16 to 18 lines, and possibly 20 or more for berthing at exposed locations. In general, it is desirable to moor a ship with the minimum number of lines practical for the given conditions. Table 6-2 gives examples of mooring line sizes, MBL, and number of lines commonly carried for a range of sizes of representative vessels. More in-depth treatment of lines carried by ships and mooring line arrangements is provided by Clark (2009) and OCIMF (2008).

6.4 Mooring Hardware and Equipment

Fenders are an important part of the overall mooring system and are covered in more detail in Chapter 5. The compression of fender systems in general helps to reduce peak loads from sudden short-duration excitations, such as waves and wind gusts, although under certain circumstances (because resilient fenders act like compression springs), they may increase periodic loads if the combined spring

Table 6-2. Mooring Lines Carried by Ships

Ship Size and Type	Line Size and Type	Minimum Breaking Load	Number of Lines
2,000 DT, general cargo	1–1/2” ± PE or PP	40–65 kips	6–8
7,000 DT, feeder container	1 1/2” to 1 5/8” PE DB or 2” to 2 1/4” PE 8 SD	65–90 kips	8
16,000 DT, general cargo container	1 3/4” PE DB or 1” to 1 1/8” WR	90–110 kips	12
70,000 DT, product tanker	1 1/4” WR	145–160 kips	12–14
160,000 DT, tanker bulk carrier	1 3/8” to 1 1/2” WR	200–220+ kips	16

Notes: Table compiled in part from information in Clark (2009) and OCIMF (2008). Values are intended to be representative. Actual lines carried and number deployed may vary widely with ship and berth conditions. PE = polyester, PP = polypropylene, DB = double braid, SD = strand, and WR = wire rope. WR may be replaced by HMPE of similar size on some vessels and may be fitted with nylon or PE tails.

stiffness of the total mooring system approaches that of the excitation. Bruun (1981) makes a case for the use of nonrecoiling-type fenders to avoid the problem of resonance and reduce vessel movements. Fenders must be able to resist peak mooring loads and to distribute them effectively into the pier structure. As a general rule, the compression of a resilient fender unit under maximum design mooring loads should not be more than about 50% of the maximum rated deflection in order to allow for unequal load distribution among fender units and additional capacity to absorb peak overloads.

Mooring hardware includes bollards, bitts, cleats, chocks, fairleads, pad eyes, hooks, and chafe guards. Bollards, bitts, and cleats are used for securing mooring lines, and normally are made of cast steel, such as ASTM A27 Gr 65-35 through Gr 40-70 or A148 Gr 80-50, or more commonly today from ductile iron, sometimes referred to as spheroidal graphite iron, conforming to ASTM A536 Gr 80-55-6 or 65-45-12. Grey cast iron may be found in older vintage structures, but it is more brittle and has lower strength and impact resistance than ductile iron, although it does have good corrosion resistance. Mooring hardware is typically secured to the structure deck with high-strength steel bolts, which may be specified under ASTM F1554 (Gr 105 is commonly used). Stainless steel bolts may also be specified in more aggressive environments. Larger bollards and bitts may be filled with concrete after installation. Bolts are often cast in place, but they may be installed through pipe sleeves cast through the pier deck to facilitate inspection and replacement. Their top exposed ends must be protected and are often run in with lead after proper tightening. See Section 7.8 for discussion of bollard anchorage design and details and Fig. 7-34 for a summary of forces acting on a typical bollard installation. Fig. 6-7 illustrates some representative mooring hardware. Additional information on a wide variety

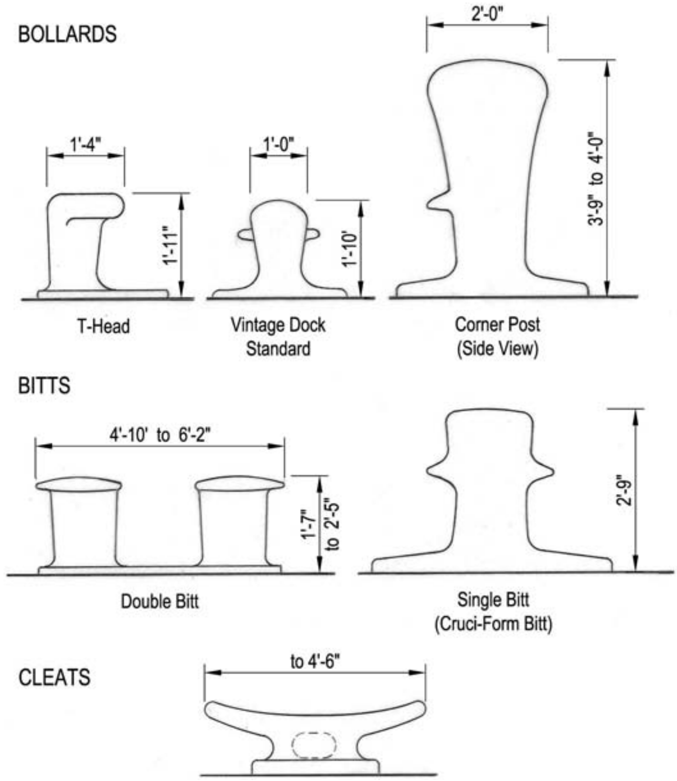

Fig. 6-7. Typical mooring hardware

of shapes and sizes can be found in the product literature of the various manufacturers and suppliers, and further description of mooring hardware in general can be found in DOD (2005a, b).

All mooring fittings should have a suitable protective coating. Cleats and hardware are often supplied as hot-dipped galvanized. Protective coatings must provide good abrasion resistance as well as long-term corrosion protection. Polyurethane systems offer superior abrasion resistance and make good finish coats. Epoxy systems may also be specified, sometimes in combination with a polyurethane top coat. Application of an inorganic zinc primer is generally recommended. Bollards and bitts can be supplied fully coated or may have the finish coat applied in the field to cover any areas exposed during installation.

Mooring hardware and equipment is normally rated for a safe working load (SWL) based upon allowable stresses and/or manufacturer's testing that should not be exceeded. The SWL should at least equal or exceed the MBL of the strongest line that is secured to it times the number of lines. Often, at facilities servicing a wide range of vessels, the exact line size may not be known for certain. In such cases, a mooring analysis can determine the maximum required line size and ultimately the required SWL of the mooring hardware. The SWL varies with mooring line angle, and most hardware is typically not load rated for vertical angles much beyond 45 to 60 degrees from horizontal. There may be directional restrictions within the horizontal plane as well.

Bollards may be used for snubbing and checking a vessel's motion, as well as for tying up. Bollard capacities are usually in the range of 40 to 100 tons of horizontal load, but they are available up to 300 tons or more. Often, larger bollards, called corner posts, are placed at the extreme ends of piers and wharves. Smaller bollards and/or bitts may be spaced approximately 40 to 100 ft apart, often alternating with cleats in between. T-head-type bollards are common on contemporary piers and wharves and can accommodate relatively steep vertical angles when such line leads are unavoidable. Although they can accommodate loads normal to and almost parallel to the pier face, they cannot take loads from the opposite direction, which may be an important consideration on piers where it may be desirable to place a mooring line across the pier under storm conditions.

Bitts may be single or double, referring to the number of posts or barrels, which usually have caps or horns to prevent lines from riding up. Bitts are used for tying up a vessel and have typical capacities on the order of 10 to 100 tons horizontal.

Cleats are used for tying up, usually for smaller vessels and lighter lines. Cleats are often not load rated and typically range in overall length over the horns from 12 in. for small craft up to 54 in. The U.S. Navy has adopted specific 30- and 42-in. cleats with 10-ton and 20-ton nominal working loads, respectively, as standards (DOD 2005a).

Pad eyes of various dimensions sometimes are provided for the attachment of chain or wire rope. Mooring rings may be provided along the face of quays where small craft are likely to tie up. *Rings* should be of at least 1-in.-diameter solid steel rod and should be let in flush with the quay face where possible.

Chocks and *fairleads* are devices used to change direction or to guide lines between points of attachment. They may be open at the top, closed, or fitted with an opening latch bar. Some also are equipped with roller guides mounted either vertically or horizontally, or both, to reduce friction and chafe.

Quick-release hooks (QRHs) are most often used at isolated mooring dolphins, especially at large tanker and gas carrier berths (Fig. 6-8). They provide added safety and security in releasing heavily loaded lines quickly. Release hooks are available with multiple hooks, and capacities range from 40 to 200 tons per hook, with total capacities up to 600 tons or more. Up to four hooks per unit are standard, but more hooks can be provided. These devices are swivel mounted with varying degrees of movement, allowing the hook to align itself with the load. In general, only one line per hook should be allowed. Where multiple lines are used, the combined MBL of all the lines should not exceed the SWL of the hook. MOTEMS (2011) requires a minimum of three hooks at breast line locations for vessels >50 kDWT and two at breast line locations for <50 kDWT at new MOTs.

Fig. 6-8. Quadruple quick-release hook at LNG terminal

Source: Photo courtesy Trelleborg Marine Systems

For multiple-hook installations, the load on the mooring structure to which the hook unit is connected may be taken as less than the sum of the number of hooks all fully loaded simultaneously. Some terminal owners and operators have their own methods of determining the number of lines to be taken as fully loaded up to their MBL simultaneously, as well as appropriate load factors for QRH anchorage design because of the generally low probability of all lines being at MBL simultaneously. Load monitoring devices equipped with threshold alarm systems also are available and can be wired to a central console so that proper adjustments can be made if required. PIANC (2012a) and BSI (2014) provide further general discussion of QRH applications, and Iversen (2014) provides in-depth treatment of structural considerations and factors of safety for QRH installations.

A variety of powered equipment may be used for mooring. *Capstans* are powered devices consisting largely of a vertically oriented rotating drum used to haul in mooring “messenger” lines from a vessel. Capstans are often provided integral with quick-release hooks, as seen in Fig. 6-8, and are common equipment at dry docks, where a vessel must be carefully hauled into a controlled position over keel blocks. Capstans typically have hauling capacities on the order of 5 to 20 tons, with variable hauling speeds of 30 to 50 ft/ min being common up to around 100 ft/ min.

Winches also are rotating drum devices, most commonly used with wire rope, which is spooled onto the winch drum. Winches are common shipboard equipment, as described in the following paragraphs. Electric-powered winches also may be located ashore or on berthing structures to provide shore augmentation. In this instance, the shore-based mooring line normally is hove out to the vessel by messenger after the vessel has been safely berthed. Shore-based winches may offer greater control of vessel mooring lines while in berth, but they also may result in an undesirable division of responsibility for the tending of lines (Bruun 1984). Capstans and winches should have reversing capability and electrical/fire hazard rating appropriate to the facility type and operating conditions.

Shipboard fixtures and equipment include chocks and fairleads, as well as bitts, cleats, capstans, and winches for handling lines. Mooring lines may be secured, "made fast," directly to the vessel's mooring hardware or may be wrapped around the drum of a winch, which in turn may be equipped with a winch brake for holding the line up to some preset tension limit or may be self-tensioning to maintain the line at some preset tension. Winch brakes have a rated holding capacity with a single layer of line around the drum of 80% MBL per ISO standards, although they are normally set at 60% MBL in service. Larger vessels may be equipped with self-tensioning winches that can be set to maintain a constant line tension. These winches work well under relatively steady load conditions, especially when pretensioned against compressed fenders; however, they may become problematic under unsteady loads, such as increasing winds, and with longitudinal forces and are therefore prohibited at most oil and gas terminals. Self-tensioning winches should have a suitable "dead band" range of tension over which it will not pay out or haul in in order to preclude excessive responses and cumulative line length changes under continual relatively small changes in line tension. Detailed description of shipboard equipment can be found in Clark (2009) and OCIMF (2008).

Mooring *line load monitoring* systems are commonly used at large tanker and gas carrier facilities (Wilson and Toth 2004). Line tensions are most commonly measured by load cells located in the pivot blocks of QRHs and can be read at remote display locations that have remote release capability as well. They can be used in conjunction with docking aid systems (see Section 5.4) that are equipped to measure fender compressions to give a complete mooring load status readout.

Environmental monitoring includes real-time wind, wave, and current measurements that can be compared with prescribed operating limits. PIANC (2012b) describes the acquisition and use of meteorological and oceanographic (metocean) data in detail.

6.5 Wind Forces

Static wind and current loads for steady flow result primarily from velocity forces that arise because of flow separation and wake formation, resulting in pressure differences on the upstream and downstream surfaces and can be calculated from the well-known drag force equation:

$$F_D = \frac{\gamma}{2g} C_D V^2 A_P \tag{6-7}$$

where
F_D = drag force,
γ = unit weight of fluid,
g = acceleration of gravity,
V = velocity of the fluid relative to the object,
A_p = projected area normal to the direction of flow, and
C_D = drag coefficient.

The drag coefficient (C_D) itself is dependent upon the object's shape and orientation to flow, surface roughness, Reynolds number (N_R) (see Section 6.10), proximity to other objects, and boundary conditions. Often, the problem of calculating static wind and current loads for design purposes can be reduced to one of selecting an appropriate drag coefficient.

For the wind speed (V_w) in knots, F in pounds, and A_P in square feet, Eq. (6-7) reduces to

$$F_w = 0.0034\ C_D V_w^2 A_P \tag{6-8}$$

for $\gamma = 0.0765\ \mathrm{lb/ft^3}$ at standard temperature of 59°F and sea level pressure. Note that air density increases almost linearly with decreasing temperature, which results in an almost 13% increase at 0°F. The increase in air density should therefore be considered at cold regions sites. Variations with moisture content and pressure are less noticeable.

The moisture content per se does not have an important effect on air density because it relates to most drag force calculations, but some investigators have questioned the effect of entrained water under storm spray and heavy rain conditions. Even a small volume of entrained water has the potential to increase theoretical wind loads by an order of magnitude. However, other investigators have argued that the water would slow down the air, thus maintaining a constant level of kinetic energy. The issue remains somewhat unresolved at this time, but it is not customary to consider entrained water effects in standard drag force calculations. Wind force calculations that are carefully carried out in accordance with the following procedures generally can be considered as accurate to within 10% to 15%.

Wind force calculations normally are carried out by resolving the wind force into longitudinal (F_{wx}) and lateral (F_{wy}) force components and a yawing moment (M_{yw}) that tends to rotate the vessel about a vertical axis through its center. Accordingly, we can write the following expressions:

$$F_{wx} = 0.0034\ C_{Dx} V_w^2 A_x \tag{6-9}$$

$$F_{wy} = 0.0034\, C_{Dy} V_w^2 A_y \tag{6-10}$$

$$M_{yw} = F_{wy} \mathrm{LOA}\, C_{ym} \tag{6-11}$$

for F in pounds, V_w in knots, A in square feet, and LOA (or LBP, depending upon the data source) in feet, and where C_{Dx} and C_{Dy} are the drag force coefficients in the respective directions, and C_{ym} is the yaw moment coefficient, all of which are functions of the relative wind direction or angle of attack (θ) (see Fig. 6-1). A_x and A_y are the end-on and side-projected areas of the vessel, respectively, and include the area of masts, stacks, rigging, deck cargoes, and so on. The yaw moment is given in terms of the lateral force times the vessel's length overall (LOA or LBP) times C_{ym}.

Figs. 6-9 and 6-10 illustrate the usual range of values and variation with wind direction of the coefficients C_{Dx}, C_{Dy}, and C_{ym} for typical merchant vessels and for vessels with boxlike superstructures, respectively. Various wind tunnel studies have been undertaken on a wide range of vessel types, and wind coefficient data can be found in the following references: for U.S. Navy vessels, DOD (2005a), which provides data based upon results of earlier tests reported by NAVFAC (1968) and Ayers and Stokes (1958); for tankers and gas carriers, OCIMF (1994, 2008) and Benham et al. (1977); for a variety of cruise ship profiles, Rice and Seelig (2010); and for various merchant vessels, Aage (1971), Altman (1971), Gould (1967), Isherwood (1973), Martin (1980), Owens and Palo (1981), Palo (1983), and Shearer and Lynn (1960). Note well that the typical values for C_{Dx}, C_{Dy}, and C_{ym} given herein are provided for general information purposes, and the designer should peruse the literature to determine appropriate values for specific applications. Wind coefficient data also may be presented in different formats, which must be verified before substitution into Eqs. (6-9), (6-10), and (6-11).

For typical oceangoing vessels, values of C_{Dy} for the wind abeam condition ($\theta = 90°$) range from 0.6 to 1.4; 0.8 to 1.0 are most representative. Occasional higher values have been reported. For wind ahead or astern conditions, C_{Dx} typically ranges from 0.4 to 1.2; 0.7 to 0.9 are representative for most vessels. In general, C_{Dx} tends to be slightly less with the wind astern than for the wind ahead. In the absence of other data, it can be assumed that C_D values vary sinusoidally between maximums at 0° and 180°, with maximums at 90° and 270° for F_{wx} and F_{wy}, respectively. Note, however, that the maximum F_{wx} and F_{wy} do not necessarily occur at these positions. For example, for most vessels with boxlike superstructures, such as ferries, passenger liners, and empty floating dry docks, the maximum F_{wy} occurs at $\theta = 45°$ to 60° and $\theta = 120°$ to 135°. For floating dry docks occupied with capacity vessels, the C_D values tend to resemble those of the docked vessel. Values of C_D vary with a vessel's overall hull shape and the extent, location, and configuration of the superstructure (including masts, stacks, rigging, and deck cargo). For a given vessel, C_D is also likely to vary noticeably with the ballast condition. An interesting, counterintuitive

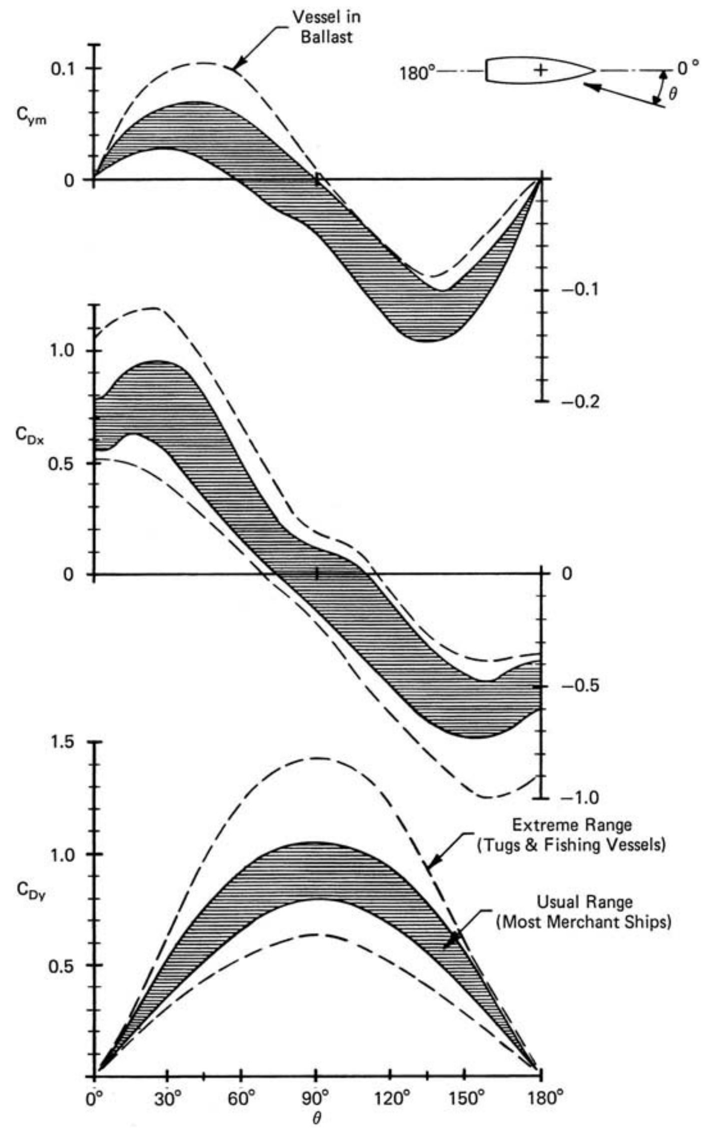

Fig. 6-9. Range of values and variation with wind direction of the lateral and longitudinal force coefficients and yaw moment coefficient for typical merchant vessels

effect occurs on large tankers with cylindrical bow shapes in ballast condition with the wind direction approximately 60 to 80 degrees from ahead: There is actually a component of the resultant force that pulls the vessel ahead, toward the wind direction, as well as pushing the bow away from the wind.

For the case of similar size vessels moored almost alongside one another such that one is directly downwind, it is often assumed for design purposes that the wind load on the downwind, shielded vessel is reduced by 50% to 80%. The shielding

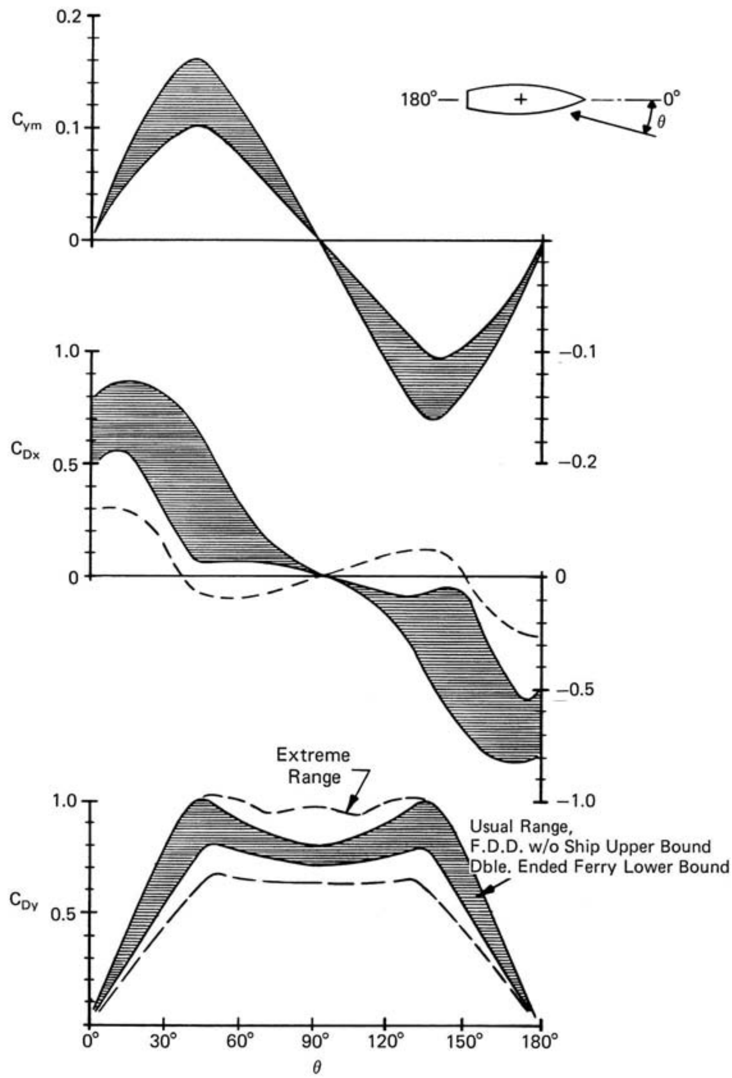

Fig. 6-10. Range of values and variation with wind direction of the lateral and longitudinal force coefficients and yaw moment coefficient for typical ferries, liners, and floating dry docks

effect varies importantly with the separation distance as well as relative vessel size. If the shielded vessel is much larger than the upwind vessel, then this assumption should not be considered valid. Seelig (1997) provides detailed treatment of wind and current forces on multiple U.S. Navy ships based in large part upon wind-tunnel model tests that have been carried out for two and more rafted vessels of various types (Ayers and Stokes 1958, NAVFAC 1968). One interesting and counterintuitive result is that, for the case of two closely moored similar vessels with wind directly abeam, the total lateral force is less than for a single vessel. The lateral force and yaw moment increase sharply with wind angle of attack off of abeam. Uhlmann (2000) presents experimental results for multiple rows of small craft in marinas and provides some discussion of the shielding effect and drag coefficients used in marina design. His results indicate that drag coefficients and shielding factors as currently used in marina design, as referenced in Section 9.7, may be conservative. For vessels moored alongside solid piers or wharves, the projected wind area can be reduced for the shielding effect of the pier or wharf for winds blowing in the off-wharf direction.

$M_{yw}(\theta)$ typically has maximum values for θ at 30° to 60° and 120° to 150°, with C_{ym} ranging from around 0.05 to 0.15. In the absence of other information, for preliminary estimates, the maximum M_{yw} can be taken at around 0.10 at 45° and 135°, varying sinusoidally in between. Note that for vessels with large superstructures located aft or for other asymmetrical configurations, the value of $M_{yw}(\theta)$ may be much less for wind forward of the beam than for that aft of the beam. The yaw moment is the result of the distance between the center of applied wind pressure and the vessel's center of lateral resistance, which are constantly changing with the angle of attack. Another important factor affecting the vessel's relative response to wind and current forces and the separations of their lines of action is the ratio of above-water to below-water areas. This ratio typically varies from around 0.4 for fully laden tankers to greater than 3.0 for passenger and roll-on/roll-off (Ro/Ro) vessels with high deck cargoes as applied to their lateral profiles (Martin 1980).

The lateral wind force causes a vessel to list away from the wind. The wind heeling moment (M_{hw}), which is resisted by hydrostatic restoring forces and/or mooring system components, is given by

$$M_{hw} = F_{wy} h C_{hm} \cos^2\psi \tag{6-12}$$

And the restoring moment (M_{rh}) is given by

$$M_{rh} = \Delta \mathrm{GM} \sin\psi \tag{6-13}$$

where

Δ = vessel displacement (DT),
GM=vessel metacentric height (see Section 9.2),
h = vertical distance from the center of wind pressure to the center of lateral resistance (CLR) of the vessel's underwater profile, and
ψ = angle of heel.

Values of C_{hm} for most vessels of interest herein range from 0.85 to 1.40. A reasonable first estimate can be made by assuming h as the distance from mid-draft to center of wind area and taking $C_{hm} = 1.0$. The wind heeling moment is not normally considered a significant factor in most mooring load calculations, although it can be readily accounted for in computer programs.

Accurate wind area information for specific vessels is often difficult to obtain. Wind profile areas for selected vessels can be found in Thoresen (2014), OCADI

Table 6-3. Approximate Lateral Wind Areas for Selected Vessels

Vessel Type	Size (DWT)	Projected Side Area (A_y) (ft^2)
Tanker	5,000	7,668[a]
	20,000	16,712[a]
	50,000	27,968[a]
	100,000	41,290[a]
	200,000	60,956[a]
	300,000	76,557[a]
General cargo (class)	5,000	9,716[a]
	10,000	14,059[a]
	12,900	24,000[b]
	20,000	20,342[a]
	30,000	25,249[a]
Containership	10,000	18,292[c]
	20,000	33,356[c]
	30,000	47,344[c]
	50,000	65,636[c]

[a]Vessel in ballast condition, calculated after PHRI (1984).
[b]Vessel in light ship condition.
[c]Vessel with containers on deck, after Thoresen (2014).

(2009), and NAVFAC (1986b). Approximate areas for selected vessel types and sizes are presented in Table 6-3. PIANC (2002) includes tables of generic vessel data, including wind areas for various vessel types within specified confidence limits. For larger vessels, such as tankers and bulk carriers, wind areas can be reasonably well correlated with displacement or DWT (BSI 2013, OCADI 2009). Thoresen (2014) cites approximate values for design estimates ranging from 22 to 36 ft^2/ft for a 30,000 DWT general cargo vessel in laden and light condition, respectively, to 63 ft^2/ft or more for large passenger vessels.

Wind loads also may be evaluated directly from the results of scale-model tests, such as the example curves shown in Fig. 6-11. Such data can be scaled to different wind velocities or to estimate loads on smaller or larger vessels of similar type (NAVFAC 1968). When test results are presented as actual measured loads on a model, care must be taken in applying the laws of dimensional analysis and similitude and in accounting for other possible scale effects when converting data to scaled prototype forms.

Reliable data from actual field measurements are relatively scarce. Fang (1979) reported on an experimental study to verify predicted wind and current loads on VLCCs, and the test results demonstrated the accuracy of the original OCIMF (1977) methodology. Fang notes that the shortest gust that fully excited the moored tanker was 20 s, and the maximum response was induced for an approximately 60-s-duration gust.

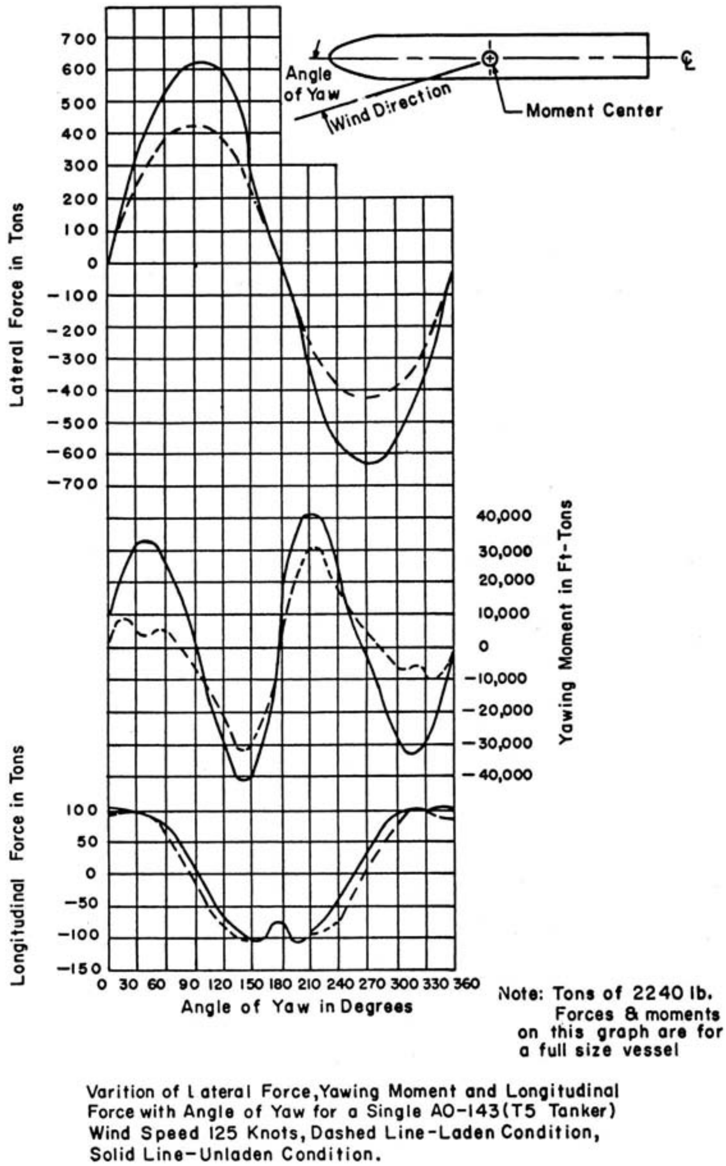

Fig. 6-11. Example model test results to determine longitudinal and lateral forces and yaw moment caused by wind on a loaded and ballasted tanker

Source: Reproduced from NAVFAC (1968)

Wind Speed Adjustment Factors

As with berthing loads, wind forces are proportional to the square of the velocity, and the determination of appropriate wind velocity is an important factor. Design wind speed may be determined by some threshold velocity, beyond which it is assumed that the vessel would not remain alongside, or on a statistical return period basis as described in Sections 6.1 and 3.5. Design wind speeds must be corrected for height and duration. If the design wind speed is based upon records at an inland site (airports are a common source of such data), then it must be further corrected for overland–overwater effects. This correction usually is on the order of a 10% increase for the harbor site, depending upon the proximity of the measurement site and exposure of the harbor. At coastal locations, the vertical profile of wind speed is typically assumed to vary roughly as the one-seventh power of the height above water. The standard reference height for anemometer records is 10 m (33 ft) above the surface. Wind speeds at other heights may be corrected to the velocity at 33 ft (V_{33}) from the velocity at elevation z, $V(z)$, according to

$$V(z) = V_{33}\left(\frac{z}{33}\right)^n \tag{6-14}$$

where the value of the exponent (n) can be varied to account for surface roughness, and is usually taken as $1/7$ for coastal sites to $1/8$ to $1/11$ over open water at offshore and exposed sites. Hsu (2002) reports that for $V_{33} > 20$ m/s, n is typically around 9 to 10, with occasional extreme values of 7 to 8 based on offshore buoy data recorded during Hurricane Kate in 1985. For most vessels of interest herein, the center of wind pressure is near the standard reference height, and applying the reference height wind velocity as an average over the vessel's height above water is often sufficiently accurate. For small craft and low-profile vessels such as submarines, resulting wind force estimates are conservative. For vessels with high superstructures extending 35 to 40 ft or more above the water surface, the height correction should be applied as described in DOD (2005a).

Maximum wind speed data often are recorded as peak gust or other averaging time, such as 1 min or more, or from older records as fastest mile wind speeds, the latter referring to wind speeds sustained long enough to cover a gust length of 1 mile. According to OCIMF (2008), a gust of 30 s is required to force a response of a VLCC in light condition on an all-wire mooring, and a gust of 60-s duration is required to force a response of a fully loaded VLCC. NAVFAC (DOD 2005a) has adopted the 30-s wind speed duration for naval vessel mooring design. Table 6-4 gives the approximate conversion factor, C_{tw}, for converting wind speeds of any duration to the 30-s gust equivalent. The correction factors apply to wind data where the mean speed and direction do not vary rapidly with time, defined as "stationary" winds. The correction factors are different for winds in hurricane-prone regions such as the U.S. Gulf and East Coasts. Refer to the wind loads commentary of ASCE 7-10 (ASCE 2010) for discussion of hurricane and nonhurricane wind zones. Nonstationary, rapidly changing wind speeds and directions may occur in extreme events, such as hurricanes, tornadoes, and gust fronts associated with any such intense storms including severe thunderstorms. Facilities designed to survive such conditions require more in-depth investigation of the wind field and are likely to be subject to dynamic analysis

Table 6-4. Wind Duration Correction Factor Relative to 30-s Gust Speed

Wind Duration (s)	Correction Factor (C_{tw}) Nonhurricane/Hurricane
1	1.182/1.221
3	1.145/1.175
10	1.080/1.097
30	1.00
60	0.938/0.950
400	0.815/0.780
1,000	0.783/0.739
3,600	0.753/0.706

Source: Excerpted from DOD (2005a).

as required by DOD (2005a). Hsu (2002) reports on gust factors during hurricanes and tentatively concludes that a ratio of peak gust to sustained wind speed of 1.3 should be conservative for most operational purposes. More rigorous treatment of wind data and wind speed design criteria, as well as bluff body aerodynamics, can be found in Simiu and Scanlan (1996) and ASCE (2010, 1987).

Dynamic Wind Forces

Unsteady wind forces, turbulence, and gustiness may result in the significant dynamic response of moored vessels under certain conditions. Of particular interest are “gust fronts,” such as may occur in thunderstorms and downbursts, and severe storms such as hurricanes/typhoons that manifest themselves in rapid changes in wind speed and direction. Vessels moored at single-point moorings (SPMs), multi-buoy moorings (MBMs), and spread moorings are especially sensitive to such conditions. Seelig and Headland (1998) report on the dynamic analysis of a ship moored both alongside a pier and at an SPM for comparison and note that dynamic loads can theoretically be up to an order of magnitude higher than those computed by static analysis based on the peak wind speed. The behavior of a large tanker moored at an SPM subject to unsteady wind, current, and wave forces was reported on by de Kat and Wichers (1991). The tanker was found to be most unstable in the ballast condition under unsteady wind and not sensitive to wind in the fully loaded condition. Fang (1979) describes field measurements of a VLCC at a marine terminal subjected to wind gusts. Typical ranges for large vessels’ resonant responses are in the range of 30 to 120 s.

Wind turbulence can be represented by a wind energy density spectrum, of which there are several formats. Most spectral formulations, such as that of Davenport (1967), were developed for land-based applications and are defined in terms of a mean wind speed and some form of turbulence intensity factor. As modified by Harris (1971), it may be applicable to in-harbor facilities. The Ochi and Shin spectrum (1988) was developed for offshore applications and maintains a

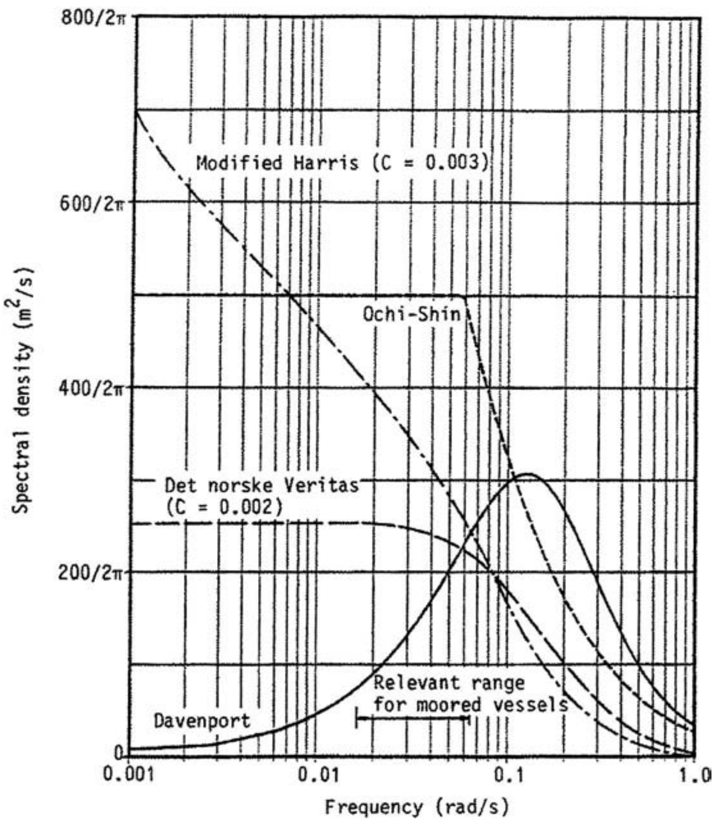

Fig. 6-12. Wind spectral densities. Comparison of Ochi/Shin, Davenport, and other spectra for 60-knot wind speed. C = wind surface drag coefficient

Source: Fiekema and Wichers (1991); reproduced with permission from the Society of Petroleum Engineers

higher energy content at lower frequencies within the range of responses from larger vessels. Fig. 6-12 shows a comparative plot of Ochi/Shin, Davenport, and other wind spectrums for an average 60-knot wind speed. For a more rigorous treatment of wind turbulence, spectra, and their application to moored ships, see Fiekema and Wichers (1991), de Kat and Wichers (1991), and Forristal (1988). Section 6.7 provides additional discussion of dynamic analysis, and Section 9.7 provides additional description of SPMs and spread moorings under combined wind and current loading.

6.6 Current Forces

Steady current forces, like static wind forces, may be calculated using the drag force [Eq. (6-8)], and similarly can be resolved into longitudinal (F_{cx}) and lateral (F_{cy})

force components and a yaw moment (M_{yc}). Using foot-pound units and taking the unit weight of seawater as 64 lb/ft^3, and with the current velocity (U_c) in knots, we can write

$$F_{cx} = 2.86\, C_{cx} U_c^2 A_{cx} \tag{6-15}$$

$$F_{cy} = 2.86\, C_{cy} U_c^2 A_{cy} \tag{6-16}$$

$$M_{yc} = F_{cy} \mathrm{LWL}\, C_{ymc} \tag{6-17}$$

where

C_{cx}, C_{ry}, and C_{ymc} = respective forces and yaw moment coefficients, and
A_{cx} and A_{cy} = projected underwater areas normal to the flow for head currents and beam currents, respectively.

For design purposes, A_{cx} can usually be taken as the product of the draft times the beam at the waterline ($D \times B$) for most ships, and A_{cy} as the product of the draft and length on the waterline ($D \times \mathrm{LWL}$). For vessels such as small craft with reduced-area midship sections, A_{cx} should be determined by applying a midship area coefficient as described in Section 2.2. Fig. 6-13 illustrates the usual range of drag and moment coefficients for deep water where the water depth to draft ratio (d/D) is greater than approximately 6. The lateral force coefficient in particular is dramatically increased by shallow water depths with decreasing d/D, and a correction factor must be applied, as shown in Fig. 6-14. The longitudinal force increase with increasing d/D is generally considerably less than the lateral force increase. It is also important to note that the lateral force component increases rapidly with current angle of attack from the bow such that it may be significant even at angles >10° with low d/D. Therefore, some variation of incidence angle, such as around 15° minimum, should be considered in design even though the vessel is berthed essentially in line with the current. Many mooring breakaway incidents have been associated with vessels moored alongside in strong currents (Clark 2009).

Current forces calculated using Eqs. (6-15) through (6-17) may be subject to greater uncertainty than wind loads calculated in the same manner. Several important factors make accurate calculation of current forces more problematical. The Reynolds number (N_R) of flow (see Section 6.10) for ships in currents normally is within the range of turbulent flow but near enough to the transition zone that a wider scatter of the force coefficients is observed (Hunley and Lemley 1980). Flow-regime effects also affect the extrapolation of scale-model test results. Full-scale measurements of current forces on a moored tanker and destroyer, as described by Palo (1986), have challenged the adequacy of conventional methods of calculating

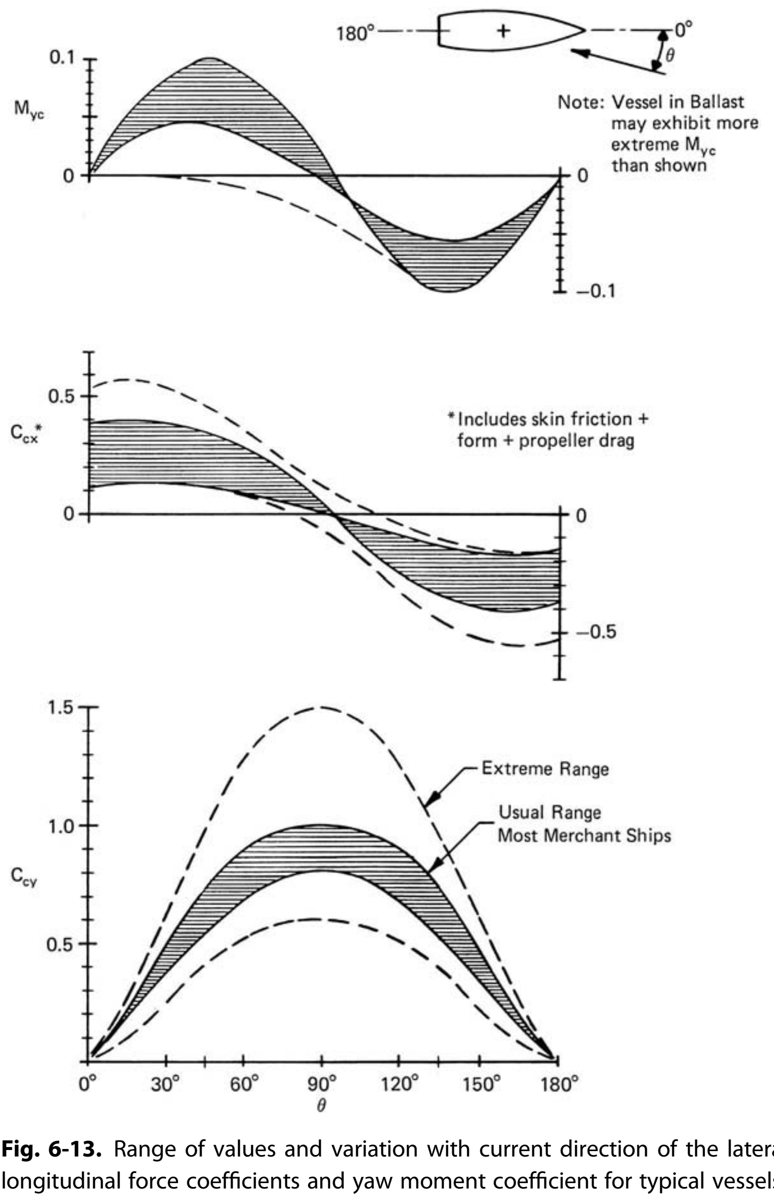

Fig. 6-13. Range of values and variation with current direction of the lateral and longitudinal force coefficients and yaw moment coefficient for typical vessels in deep water

current loads. Some important conclusions from Palo's work are that the lateral force coefficients are relatively insensitive to hull shape but show a significant dependence upon the vertical distribution of current velocity, also referred to as current shear. Palo further notes that the usual shallow water correction factors do not adequately represent actual shallow water behavior, but without further actual prototype testing, an improved design methodology cannot be devised. Other factors

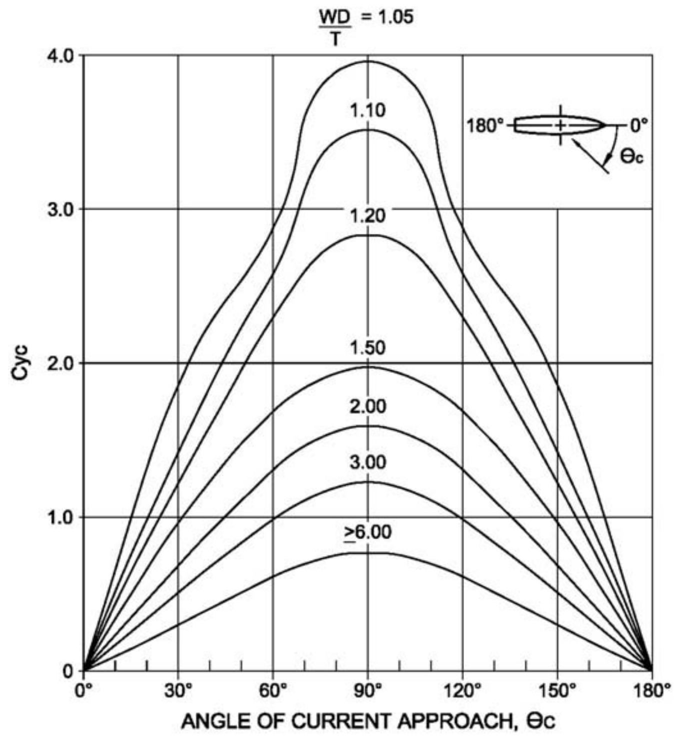

Fig. 6-14. Lateral current force coefficient versus angle of incidence for varying water depth to draft ratios

Source: NAVFAC (1982)

also make current force determination difficult: the longitudinal force cannot be accurately calculated using a simple form drag coefficient; the velocity profile around the entire vessel is difficult to assess and may be significantly affected by bottom conditions and proximity effects; current velocities are not necessarily steady; and dynamic behavior may need to be considered.

For the time being, port engineers must apply conventionally accepted methods but should be aware of their limitations and of particular situations in which they are more or less applicable. Design guidance and current coefficient data for evaluating current loads can be found in DOD (2005a) and NAVFAC (1968) for U.S. Navy ships; BSI (2014), Palo (1983), BSRA (1973), and Altman (1971) for merchant ships; and OCIMF (2008, 1994) and Benham et al. (1977) for tankers and gas carriers. Seelig et al. (1992) present a simple method for calculating broadside current forces based upon review of existing experimental data. They note that the channel effect of narrow laboratory flumes tends to artificially increase values of drag coefficients. Although this phenomenon may result in some conservatism in the design of moorings in open water, unobstructed pier location values of drag coefficients may not be so conservative in narrow channels. In general, channel effects become noticeable when the channel width is less than around five times the vessel beam, which also, depending upon water depth, restricts the channel flow and results in higher local current velocities. An in-depth treatment of hydrodynamic forces on ships' hulls and of scale-model applications is provided by Newman (1977) and Faltinsen (1990).

In deep water, overall lateral force coefficients, C_{cy}, generally range from 0.5 to 1.5; 1.0 is the approximate mean value. The lateral force varies almost sinusoidally with the angle of attack but exhibits greater variation than wind forces in this regard (refer to Fig. 6-12). As previously noted, measured lateral force coefficients show little sensitivity to hull form but increase dramatically with reduced water depth to draft (d/D) ratio. An increase on the order of six times the deep water value ($d/D > 6$) when d/D is reduced to 1.1 or less may occur. Fig. 6-14 illustrates the variation of C_{yc} with d/D and current angle of attack for generic naval vessels (NAVFAC 1982) developed from nondimensionalized model test data (NAVFAC 1968) and OCIMF test data conducted between 1968 and 1977. Note the steep rise in lateral force component with small angle of attack from ahead and astern. For this reason, some angle of attack should be assumed in mooring load calculations to account for variations in current direction, even when the vessel berth is aligned with the prevailing current direction. The maximum lateral force on a given vessel may actually occur at current attack angles other than 90°. Palo (1986) notes maximum lateral forces for $\theta = 110$.

The lateral force and yaw moment are closely related, and Palo notes maximum values of C_{ymc} of about 0.1 with a zero crossing near $\theta = 90°$ as reasonably representative in uniform current fields. Current yaw moments vary more symmetrically with direction than wind moments and, like the lateral force, are dramatically affected by the relative water depth. The ballast condition and the vessel trim also significantly affect the yaw moment and lateral force. Fig. 6-15 illustrates the variation in yaw moment coefficient with d/D and current angle of attack, which exhibits a dramatic increase with decreasing d/D similar to the lateral force coefficient and is based upon the same data as Fig. 6-14.

Longitudinal current forces are the result of both form (pressure) drag and surface friction drag and do not exhibit quite as dramatic increase with decreasing d/D as the lateral force and yaw moment components. Propeller drag is sometimes added separately to the hull form plus friction components. In reality, form, skin friction, and appendage drag interaction cannot be effectively separated, and many designers choose to apply a single overall drag coefficient for a given vessel type. Overall form-drag coefficients for F_{cx} range from around 0.10 to 1.0 (0.20 to 0.60 are more representative), depending upon the hull configuration, appendages, surface roughness (e.g., fouling can greatly increase longitudinal drag forces), angle of attack, and so on. Interestingly, vessels with twin propellers tend to have lower overall form drag coefficients than those with single screws. It should be noted also that the maximum longitudinal force may occur for angles

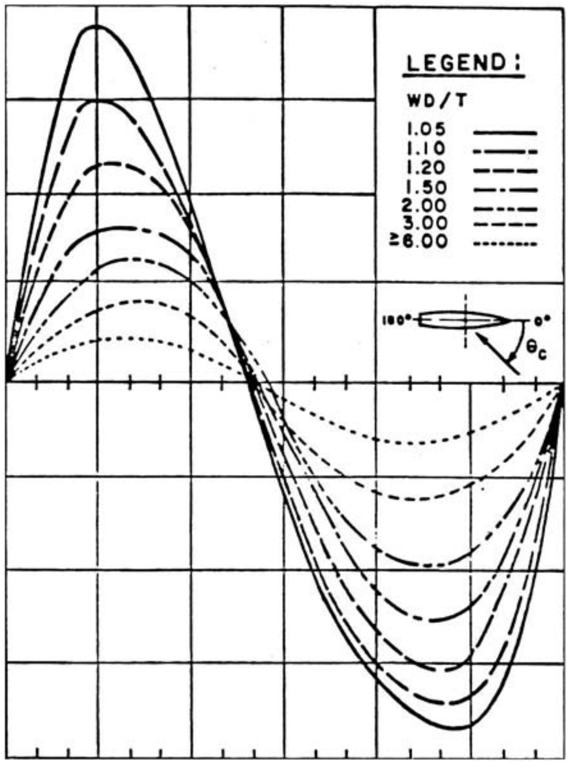

Fig. 6-15. Current force yaw moment coefficient versus angle of incidence for varying water depth to draft ratios

Source: NAVFAC (1982)

of attack other than 0° and 180°, and in fact, it often remains near its maximum to within 10° to 15° of lateral. For a full-bodied loaded tanker, for example, with wind from around 20 degrees from the bow, there is a significant force component acting to pull the ship ahead, into the current, similar to wind acting on a tanker with a cylindrical bow.

Skin friction drag coefficients are dependent upon the Reynolds number (N_R) (see Section 6.10) and for the range of interest herein may range from 0.001 to 0.01, with the higher value generally more appropriate for design. Note that the characteristic length for determining N_R for the longitudinal friction drag is the vessel waterline length (LWL), which should be corrected for angle of attack, thus, $N_R = V \times \mathrm{LWL} \times \cos\theta/\nu$. DOD (2005a) provides further guidance for determining skin friction coefficients. The skin friction term is similar to the calculation of Eq. (6-11), except that the area used is the total surface area of the underwater portion of the hull (A_s). Specific information on wetted surface areas for a given vessel may be difficult to obtain. DOD (2005a) presents the following formula for estimating the wetted surface area (A_{ws}) of typical naval vessels:

$$A_{ws} = 1.7\,\text{LWL} \times D + \Delta/\gamma D \tag{6-18}$$

Note that this equation can be rearranged in terms of the vessel's block coefficient (C_b) as

$$A_{ws} = 1.7\ \text{LWL} \times D + \text{LWL} \times B \times C_b \tag{6-19}$$

The designer with a need to more accurately determine a vessel's wetted surface and appropriate friction factors should consult the general literature on naval architecture, as referenced in Chapter 2. The form drag of a vessel's hull alone facing the current direction is relatively small. Hunley and Lemley (1980) note an average drag coefficient corresponding to C_{cx} for the hull alone of 0.088 with a standard deviation of 0.039 based upon some 60 ships from Society of Naval Architects and Marine Engineers (SNAME) resistance data sheets.

The longitudinal resistance of locked propellers can easily exceed that of the vessel's hull itself for certain vessels, such as high-speed naval craft-like destroyers. Propeller drag can be estimated by applying a drag coefficient of unity to the expanded blade area of the propeller, which is a function of the total area and the propeller pitch to diameter ratio. Obtaining propeller dimensions may be difficult. DOD (2005a) provides guidance for determining propeller areas and propeller area ratios for vessel types.

Another form of longitudinal resistance important in the design and powering of vessels is wave-making resistance. However, this form of resistance can generally be neglected for vessel mooring analysis because of the relatively low current speeds compared to the speeds the vessel can attain under power. Wave-making resistance is generally negligible when the nondimensional Froude number, N_F (see Section 6.10), is less than around 0.1 and in some cases up to 0.18. For small craft in very strong currents, say, exceeding 4 to 5 knots, wave-making resistance could become important. In such cases, it should become apparent that vessels should not be moored alongside in such conditions.

Current Speed Adjustments

For design purposes, the value of U_c used should be the average over the vessel's draft for the maximum current expected at the site. Where current velocities are significant, say, in excess of 2 to 3 knots, site-specific measurements should be taken. Current velocities may vary greatly with distance from shore, water depth, and local boundary conditions. It also should be noted that the construction of a new structure may alter existing current patterns. In shallow open water, the vertical velocity profile usually is assumed to vary (like wind) with the one-seventh power of the height above bottom and is maximum at the surface. (Refer to the discussion of currents in Section 3.4.) As previously noted, the actual vertical distribution or current shear may have a significant effect upon the forces realized. For large, deep-draft vessels in relatively shallow water, the variation of current with depth must be

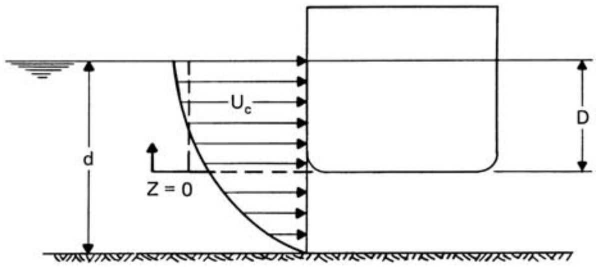

Fig. 6-16. Distribution of current velocity over vessel's draft

considered. Fig. 6-16 illustrates the distribution of current velocity normal to a ship's hull in shallow water. The averaged squared velocity of the current can be found from

$$U_c^2 = \frac{1}{D}\int_0^D (U_z)^2 dz \tag{6-20}$$

where U_z is the velocity at depth z. Assuming that the current velocity varies according to the one-seventh power law, then for a vessel with small under-keel clearance $(d/D = 1.1)$, the average velocity over the vessel's draft would be about 90% of the surface current velocity (OCIMF 2008).

Local Current Effects

In narrow or constricted channels, the current velocity profile across the channel can be affected by the presence of the vessel when the channel width (W) is less than approximately five times the vessel's beam (B). The blockage ratio equal to the vessel's underwater sectional area divided by the channel cross-sectional area provides an index of the severity of this effect. Water accelerated below the vessel's keel causes the vessel to sink deeper, termed *squat*, and also to trim, especially in shallow water and with restricted channel width. Squat is generally small for moored vessels—generally less than a few inches is normally neglected in mooring calculations—although it is of prime importance in channel layout and design.

Although currents can generally be treated as steady, uniform flow for most mooring design applications, in reality, currents, like wind, are an unsteady phenomenon. Tidal flows exhibit major changes in flow speed and direction over a period of hours, with minor fluctuations on the order of minutes. Downstream of islands and obstructions and along irregular shorelines and bottom contours, large-scale eddies may be formed that result in rapid changes of speed and direction over a few minutes or less. If the diameter of such eddies is on the order of the size of the vessel or larger, then dynamic enhancement of mooring forces and vessel motions may result. As with wind, freely moored vessels are most susceptible. Physical model tests may be required for the evaluation of such conditions. An example of such test results and their application to a tanker moored at an SPM in unsteady wind and current and wave action is provided by de Kat and Wichers (1991). A detailed analysis of current forces acting on dredging equipment is provided by Wichers (1992), and rigorous textbook treatment of both steady and unsteady current loads on ships can be found in Faltinsen (1990).

Current Standoff Forces

A vessel moored alongside a pier or quay located in a river or narrow tidal estuary with strong currents may be subject to hydrostatic pressure forces arising from an unequal velocity distribution about the vessel. A manifestation of this force is illustrated by the case of a vessel transiting a channel. If the vessel is too near a bank, the flow of water essentially channels itself around the outside of the vessel, resulting in an accelerated flow around one side, which in turn causes a hydraulic gradient across the vessel's width with higher water near the bank, thrusting the vessel away from the bank. As the vessel moves toward the center, the redistribution of flow results in a suction back toward the bank. At the center of the channel, side forces cancel, and no net suction force acts on the vessel. In the case of a vessel moored alongside in a relatively strong current, the slowing down of water passing through, or being totally obstructed by, the pier creates a velocity-induced pressure head against the vessel's side, resulting in an outward pull on the mooring lines, known as a *standoff force*. Fig. 6-17 illustrates this effect.

Khanna and Sorensen (1980) report on a design case history where model experiments were used to predict the standoff forces, which were manifested in a net sway force away from the structure, a longitudinal surge force, and a yawing moment. They also note dynamic forces in terms of coupled surge–yaw and sway motions that took place at the natural periods of the moored ship. The differential head across the vessel can be expressed by the Bernoulli equation as a coefficient (C_{so}) times the velocity head, $h = U_v^2/2g$, where U_v is the undisturbed current velocity. Actual field measurements originally reported by Jackson (1973) indicate that this head can be as high as $0.42 U_v^2/2g$. Khanna and Sorensen, however, found values of C_{sy} ranging from 0.10 to 0.15 and 0.07 to 0.12 for scaled current velocities of 5 knots and 7 knots, respectively, depending upon the berth location. Their experiments were carried out on a 1 : 100 scale model of a 300,000-DWT tanker with $d/D = 1.5$. They found that a reduction in d/D to 1.32 resulted in an approximately 35% increase in the standoff force. Motions of the vessel started when the current exceeded a threshold velocity of around 2 knots, and dynamic mooring line loads were on the order of two to three times that because of the static head. The use of more resilient, "softer," moorings has been shown to reduce peak dynamic loads (Wood 1980) and (Khanna and Ottesen-Hansen 1977); however, adequate line pretension is also necessary. The longitudinal force also was increased by the standoff effect. There was some correlation with pile area density, but a 16% increase in pile density did not alter

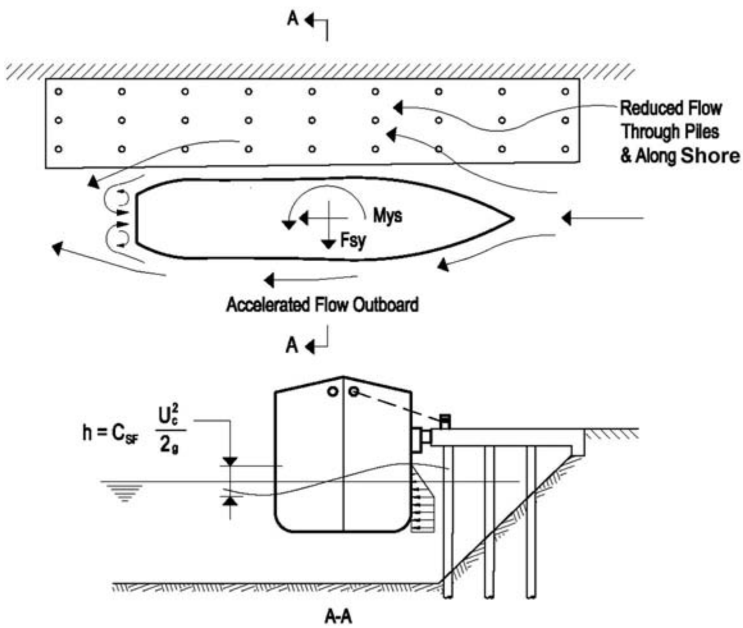

Fig. 6-17. Vessel standoff force definition sketch

Note: This figure is not to scale; relative water levels are greatly exaggerated

their results significantly. Ball and Wilcock (1981) report on follow-up studies to determine the effects of pile density, spacing, and configuration.

Combined Wind and Current Forces

In combining wind and current forces in static mooring analysis, it is generally necessary to examine various load cases, usually on a trial-and-error basis, to determine the worst-case conditions of vessel draft, water levels, and wind and current directions. Current forces are generally maximum for fully loaded vessels at low tide, whereas wind forces are higher for vessels in ballast condition at higher water levels where the pier or wharf structure may provide less shielding than at low water. In addition, current speeds are typically highest near midtide and vary with direction as well as between flood and ebb at tidal sites. The coincidence of maximum winds and currents may also need to be considered in light of long-term probability of occurrence and any correlation between high winds and strong currents.

6.7 Passing Vessel Effects

An important phenomenon is the effect of passing vessels on moored ships, especially in narrow or constricted channels where large vessels transit at relatively high speeds. There have been many serious mooring breakaway incidents that underscore the importance of understanding passing vessel effects (ASCE/COPRI 2014) and Seelig (2001b). Experiments have shown that the forces and moments induced on a moored vessel by a passing vessel, which can be significant, are more importantly related to the pressure gradient surrounding the vessel than to the waves generated by the moving vessel. Movement of water around the passing ship creates a Bernoulli effect suction on the moored ship. In general, the forces associated with surge, sway, and yaw motions are proportional to the square of the speed of the passing vessel (V_s) and a function of the separation distance and water depth to draft ratio. Other factors affecting the maximum mooring line loads are length of time for passing vessel to transit, ratio of length of moored vessel (L_{mv}) to length of passing vessel (L_{pv}), vessel ballast condition, relative displacements of the vessels, and mooring line pretensions. In general, stiffer moorings result in smaller forces. Forces on the moored vessel are greatest when $L_{mv} < L_{pv}$. The effect of water depth is minimal when d is greater than the center-to-center vessel separation distance. When currents are present, the speed of the vessel is relative to the moving water; so an opposing current increases the force, and following currents decrease it.

Early investigations were based upon theory and physical model tests and provide a means of estimating passing vessel forces and moments on moored vessels for a range of conditions. More recent studies involve computer modeling with the application of wave pressure field theory. Wang (1975) developed a theoretical method, based upon slender body theory assumptions, for calculating passing vessel forces for the deepwater case of parallel vessels and also determined correction factors for shallower water cases. Fig. 6-18 is a definition sketch for the parallel passing vessel case, and Fig. 6-19 shows a nondimensional plot of the variation of forces and moments on a moored vessel in deep water caused by a passing vessel, based upon the work of Wang (1975). Wang presented graphs of nondimensional

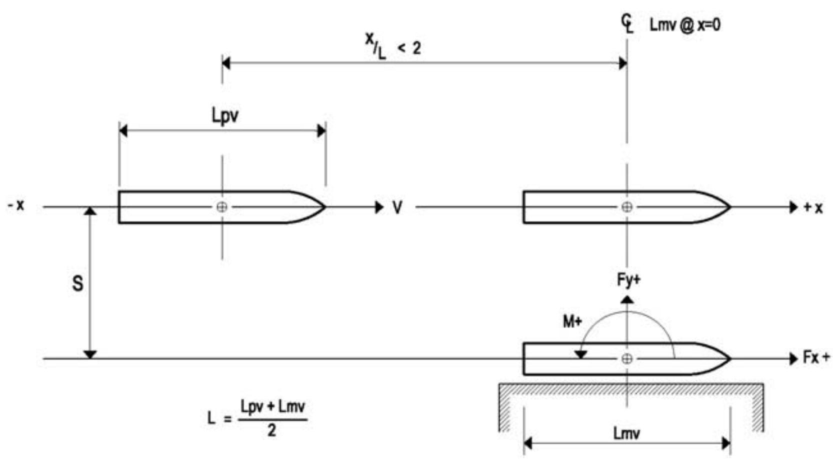

Fig. 6-18. Passing vessel geometry definition sketch

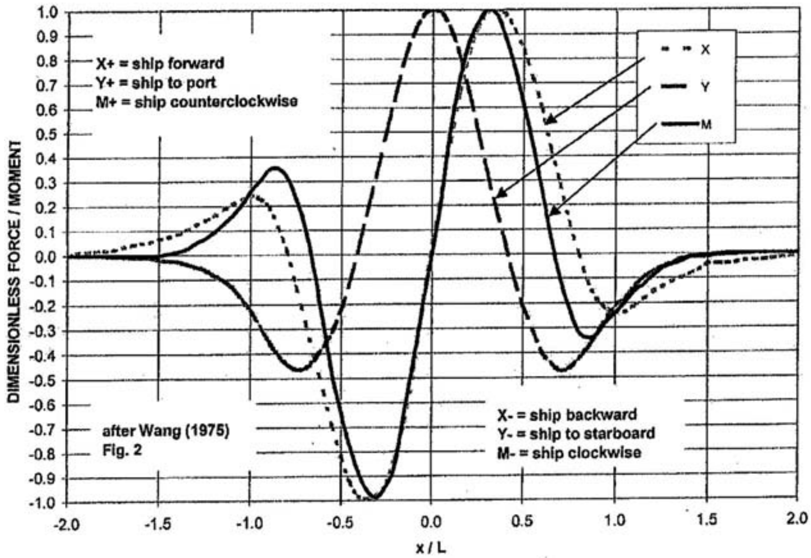

Fig. 6-19. Nondimensional graph of variation in surge and sway force and yaw moment acting on a moored vessel in deep water caused by a passing vessel

Source: Wang (1975)

surge and sway forces and yaw moments for variable vessel length and midship area ratios based upon theoretical analysis. A general narrative of the passing vessel effect is as follows for vessel center-to-center separation distance (S) generally less than one ship length (L). When the approaching vessel is two or more vessel lengths away from the moored vessel, there is virtually no effect. As the passing ship's bow reaches the stern of the moored ship, a small surge force in the direction of the passing ship and a small moment turning the moored ship stern away from the advancing vessel are experienced by the moored ship. As the passing ship's bow reaches the midship of the moored ship, it experiences a large surge force drawing it toward the passing vessel and a large yaw moment swinging its stern toward the passing vessel. When both vessels are exactly abeam of each other, a large sway force pulling the moored vessel toward the passing ship occurs. As the passing ship's stern reaches the moored ship's midship, there is a large surge force in the direction of the passing ship and a large moment swinging the bow of the moored ship toward the passing ship; and as the stern of the passing ship reaches the bow of the moored ship, a small surge force acting opposite to the passing ship direction and a small yaw moment turning the bow of the moored ship away from the passing ship are experienced by the moored ship. Seelig (2001b) developed shallow water correction factors for Wang's theory that are more applicable to typical port situations and a spreadsheet program application, PASS-MOOR to evaluate passing vessel forces and moments for a range of conditions with varying L_{pv}/L_{mv}, S/L, and D/d. Kriebel (2005) later conducted physical model tests using a Series 60 hull form (a generic cargo ship widely used in naval architecture studies) to refine the results of Seelig. Kriebel presented equations for the peak surge (F_x), sway (F_y), and yaw moment (M) felt by the moored vessel as follows:

$$F_x = C_x \times \rho/2 \times V^2 DL \tag{6-21}$$

$$F_y = C_y \times \rho/2 \times V^2 DL \tag{6-22}$$

$$M = C_m \times \rho/2 \times V^2 DL^2 \tag{6-23}$$

Kriebel tabulated the coefficients for a range of conditions with varying S/L, D/d, and displacement ratio, Δ_{pv}/Δ_{mv}. For an example of one of the more severe cases tested with $S/L = 0.4$, $D/d = 0.9$, and $\Delta r = 1.1$, the above coefficients are (rounded) $+C_x = 0.038$, $-C_x = 0.046$, $+C_y = 0.217$, $-C_y = 0.105$, $+C_m = 0.027$, and $-C_m = 0.025$. There is wide variability in the range of coefficients with high sensitivity to S/L and D/d. Kriebel also conducted similar model tests for the case of a moored ship perpendicular to the passing ship using the same format and variables in tabulating the results (Kriebel 2007). The most notable aspect of the perpendicular ships case is a large surge force pulling the moored ship toward the passing ship when the passing ship is directly abeam of the moored ship.

Other notable early investigations were based upon scale-model tests of mostly large tankers. Scale-model test results and calculation methods have been reported by Remery (1974), Muga and Fang (1975), and Lean and Price (1979). Flory (2001, 2002) reviews these past works and presents a simplified method of estimating passing ship forces, including nondimensional graphs for surge, sway, and yaw moments relative to the passing vessel position, based upon the experimental work of Remery conducted primarily on tankers. The forces and moments vary with the square of the velocity of the passing ship. The program DELPASS was developed by Pinkster (2004) to evaluate the influence of passing vessels (PVs) on ships moored in restricted waters. Flory and Fenical (2010) report on the proximity effects of quay walls and restricted channels on passing vessel induced forces.

Forces caused by passing ships are dynamic and potentially may be large. The equations presented above can be applied in a static mooring analysis to provide an estimate of the magnitude of forces and resulting line loads. The mooring loads can be more accurately determined by dynamic analysis by applying a time history of the passing vessel response for which the general nondimensional curves of Fig. 6-19 can be used, knowing the vessel's speed and hence time of transit. Smith and Headland (2004) provide an example dynamic analysis of a moored liquefied natural gas (LNG) vessel subject to passing vessel effects based upon the methodology of Seelig (2001b), and Smith et al. (2010) discuss passing vessel modeling in port design and operations. Hydrodynamic analyses of pressure field effects, including time-dependent 2D and 3D methods and Reynolds-averaged Navier–Stokes (RANS) methods (Chen et al. 2002), may be conducted at levels of sophistication appropriate to the given situation. Modeling methods based on finite difference codes have been developed by Nwogu (2007) and Fenical et al. (2006), and finite element codes have been developed by Stockstill and Berger (1999). Direct pressure field modeling allows evaluation of complex channel and bank configurations and varying vessel hull forms and other complicating factors that cannot be readily accommodated with empirical methods (Fenical 2007). Refer to (ASCE/COPRI 2014) for further discussion of hydrodynamic modeling methods. The 33rd PIANC World Congress included several papers on the joint industry project, ROPES (Research On Passing Effect on Ships), as summarized by van den Boom et al. (2014), including recent full-scale measurements, model testing, development of analytical methods and mitigation measures.

MOTEMS (2011) provides additional design guidance for PV analysis and when it is required to be carried out for California MOTs.

6.8 Wave Loads and Vessel Motions

There is no generally accepted standard procedure for calculating wave loads on fixed moored vessels. Wave forces and mooring system reactions arise from a complex interaction of the water particle kinematics, vessel motions, and mooring system response. Determinations of added mass, damping, and higher order nonlinear effects add to the difficulty. Because of this complexity, an analytical treatment is difficult and best accomplished by numerical modeling and/or scale-model testing. A brief introduction to these subjects is provided in Sections 6.9 and 6.10. This section describes the nature of wave loading and certain approaches that have been taken to the problem and presents some general findings of various investigations.

Wave action can be generally classified according to the frequency of motion as short-period wind waves and swell and long-period wave action, which is generally a result of harbor or basin oscillations known variously as seiche, surge, and ranging action. Wind waves and swell are of central importance at offshore terminals and typically have periods of less than 20 s. An important second-order nonlinear effect arising from continuous wave action against a moored object is the drift force. It consists of a steady-state component and an oscillating component (in irregular seas) associated with wave grouping action and having a periodicity on the order of 20 to 100 s, which is within the range of natural periods of large moored vessels. Long-period wave action manifests itself typically in periods ranging from around half a minute to several minutes or more and can be a serious menace even in protected harbors when the natural period of the mooring system and the seiche period are almost coincident, and/or when a vessel is located near a nodal point (see Section 3.4) in the harbor where the horizontal flow of water associated with the seiche action is at its greatest. Notable early investigations of vessel surging problems because of long-period waves have been presented by Wilson (1951, 1959) and by O'Brien and Kuchenreuther (1958). A more recent example pertaining to long-wave problems at Long Beach, California, was presented by Headland and Poon (1998). Goda (2000) provides a general textbook treatment of harbor tranquillity and general discussion of moored vessels and their response to harbor waves. Jensen and Warren (1986) provide an example of harbor wave physical and numerical modeling, and McGehee (1991) reports on field measurements of five vessels subject to long-wave excitation within Los Angeles and Long Beach, California, harbors.

If we consider a free-floating, unrestrained object, it responds to the passage of waves with some degree of motion in each of its six degrees of freedom. It is interesting to note that for the case of an unrestrained, freely floating body in long waves, the force on the body is equivalent to its own weight (displacement) sliding down the slope of the sea surface (Dean and Dalrymple 1991). The force imposed upon a mooring structure is largely a function of the degree of restraint or the effective spring constant of the fenders and mooring lines. Because waves are a time-dependent phenomenon, dynamic analysis must be applied to obtain meaningful results. In general, the force is proportional to the wave height and vessel draft for a given wavelength and water depth. The behavior of moored vessels as oscillating systems was introduced in Section 6.1 and was treated in general by Vasco Costa (1983). The natural period of a moored vessel can be increased by the use of longer and softer hawsers and softer fenders and can be decreased by the use of shorter, stiffer hawsers and stiffer fenders. This fact can be used to reduce the potential for mooring systems resonance, depending upon the prevailing wave climate at the site. For a vessel moored alongside in line with the direction of wave approach, the natural period in surge, T_s, can be estimated by treating the vessel and mooring lines as a simple single-degree-of-freedom system and neglecting damping, which is generally for low-frequency motions in surge, and further assuming no coupling with other motions as given by

$$T_s = 2\pi\sqrt{M_v/K_s} \tag{6-24}$$

where

M_v = virtual mass of vessel plus entrained water, which can be taken as = 1.15× vessel mass for surge (USACE 2002), and

K_s = spring constant of sum of mooring lines providing longitudinal restraint, i.e., spring lines with respect to their directions and assuming that they are symmetrically laid out in fore and aft restraint. K_s then equals the sum of $T_l/\Delta l$ in all of the lines for each direction [see Eqs. (6-1) and (6-2)].

Note that the mass of the vessel may change during cargo-transfer operations. The natural period of the moored vessel may also be changed by adjusting the mooring line arrangement and tensions. Fig. 6-20 illustrates a ship moored alongside

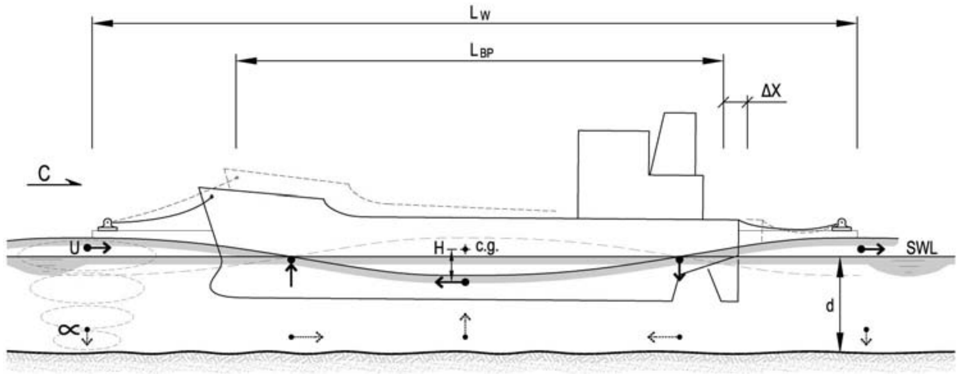

Fig. 6-20. Vessel moored in head sea. Arrows within wave profile indicate the direction of the water particle velocities (v) and accelerations (α) as the wave form passes

subject to head-on wave action. Arrows indicate the direction of wave particle velocities (v) and accelerations (α) at the given locations within the wave. As the wave crest reaches the ship's forward end, the bow pitches upward and the ship surges aft until checked by the mooring lines, and the wave particle velocity reverses with the crest at the ship's midship position with maximum upward heave. The situation reverses as the trough reaches the ship's midship and the stern lines check the forward motion. How much the ship moves and the resulting mooring line loads depend upon the height and period of the incident waves relative to the ship's natural periods in surge, pitch, and heave. Clearly, resonance must be avoided. In general, when the wave length (L_w) to ship length (L_s) ratio, $L_w/L_s > 2$, the heave and pitch response approaches unity, and for $L_w/L_s < 0.75$ is minimal.

Note that the wave particle orbits are almost circular in deep water and become more elliptical in shallower water depths. The vessel's chocks locations therefore follow somewhat elliptical to almost circular orbits as the wave form passes.

Wave forces on moored vessels manifest themselves in two components. The first is a linear oscillatory force at the frequency of the waves. This force can be found by integrating the varying water pressures around the submerged portion of the hull (Fig. 6-21). Diffraction theory and numerical analysis techniques are used in order to do this because the presence of the vessel modifies the incident wave train. The oscillating vessel scatters waves, which propagate away from the hull and contribute to the damping of the oscillations. The hydrodynamic coefficients of added mass and damping are different for each of the six degrees of freedom and are dependent upon the frequency of the oscillation. In shallow water, the water-depth-to-draft ratio (d/D) or the vessel under-keel clearance (UKC $= d - D$) has a significant effect on these coefficients. Buoyant restoring forces may result in heaving and/or pitching oscillations at some natural period of the system. The restoring forces of mooring lines and fenders are nonlinear and may result in resonant horizontal motions. Resonant response may also occur at some subharmonic of the wave frequency

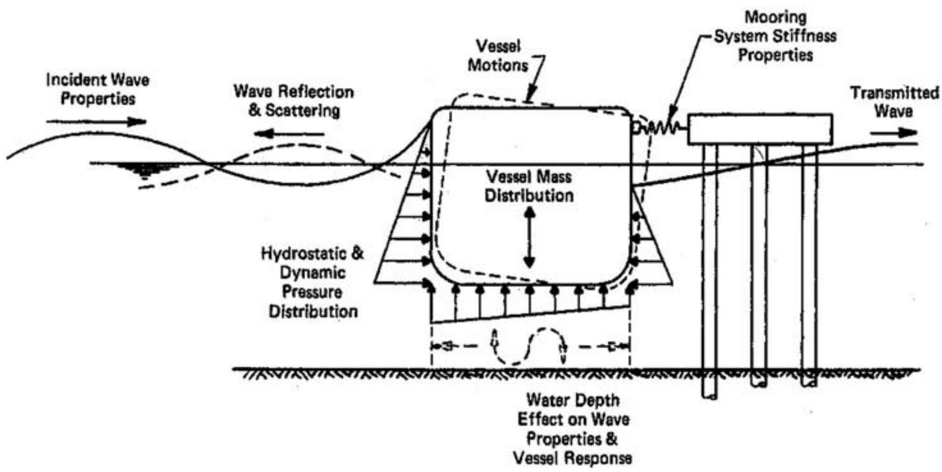

Fig. 6-21. Schematic illustration of wave force on fixed moored vessel in beam sea

(i.e., the wave period divided by an integer). Goda and Yoshimura (1971, 1972) present a method of determining the static wave force on a vessel rigidly moored at offshore (wave-transparent) dolphins by solving the general diffraction problem for an elliptical prism in shallow water. The method was proposed to estimate the total force on the dolphins for preliminary design but does not account for mooring stiffness and relative vessel motions. Textbook treatment of the general approach to solving wave–structure interaction problems for wave loads on floating bodies can be found in Newman (1977), Sarpkaya and Isaacson (1981), Chakrabarti (1987), Faltinsen (1990), and Dean and Dalrymple (1991).

The second type of wave force is nonlinear and a result of the irregular sea state. This force, known as the *drift force*, is primarily a consequence of wave grouping and setdown effects. The momentum flux and radiation stress of progressive groups of higher and then lower waves result in a net force in the direction of wave propagation. The drift force is a steady force in regular waves and a slow-varying/low-frequency force in irregular waves. Because damping is relatively low at a lower frequency and because the usual range of periods of 20 to 100 s is also within the range of natural periods of moored ships, the slow-varying drift force may cause overstressing of mooring lines and large fender forces. Analytical, computational, and physical modeling techniques used to approach the general moored ship problem are reviewed in the next section.

The drift force is generally much smaller than the oscillating wave force. In fact, van Oorshot (1976) notes that for tests carried out on a model ore carrier, the steady drift forces in regular waves were found to be on the order of 1% or less in the head sea and 2% to 4% in the beam sea condition of the harmonic oscillatory wave forces. Because of its importance in the mooring of large vessels, however, the slow-varying drift force has been studied by several investigators, such as Hsu and Blenkarn

(1970), Faltinsen and Michelsen (1974), van Oorshot (1976), and Loken and Olsen (1979). Chakrabarti (1980) reviews the work of various investigators and the theoretical development on steady and oscillatory drift forces on floating objects. The drift force is proportional to the square of the wave height and represents the excess momentum flux of the radiation stress. According to Sarpkaya and Isaacson (1981), where no energy is dissipated, the magnitude of the drift force as derived from small-amplitude wave theory can be calculated from

$$F_{dr} = \gamma \frac{H_r^2}{8}\left(1 + \frac{2kd}{\sinh 2kd}\right) \tag{6-25}$$

where H_r is the reflected wave height based upon the root mean square value of the wave spectrum = $0.707 H_s$. It is therefore necessary to determine a reflection coefficient ($C_r = H_r/H_i$) for the given sea conditions. Note that the incident, reflected, and transmitted wave heights are related by

$$H_i^2 = H_r^2 + H_t^2$$

as described in Section 9.1.

Where wave-induced vessel motions and mooring forces can be assumed to be linear and frequency-dependent, their statistical values can be determined by spectral-response analysis techniques. This determination requires that a transfer function, or response amplitude operator (RAO), which describes the vessel unit response in a given mode and at a given frequency, be determined. RAOs for specified conditions may be determined from theory or experiment as described in the following section. For a unidirectional sea spectrum, the response spectral density ($S_r(\omega)$) is obtained by multiplying each ordinate of the input wave energy spectrum, $S(\omega)$, by the square of the RAO, as given by

$$S_r(\omega) = [T_j(\omega, \theta_l)]^2 S(\omega) \tag{6-26}$$

Here, the subscript j indicates the jth degree of freedom (DOF); the subscript l indicates the direction of the jth DOF; ω is the circular frequency, $2\pi/T$; θ is the angle indicating the direction of motion; and $T_j(\omega, \theta_l)$ represents the transfer function or RAO.

Fig. 6-22 schematically illustrates the response-spectrum analysis procedure. A simple example applied to the heave motion of a rectangular barge is given in Section 9.3. Fig. 6-23 shows computed mooring line force spectra for a 7,348-ton cargo ship moored alongside an open pier. The spectra were generated by a computer program based upon a mathematical model, as described by Bomze (1980). Fig. 6-24 shows the vessel motion response spectra for surge, sway, and yaw, and comparison with spectra developed from actual field measurements conducted during field validation studies.

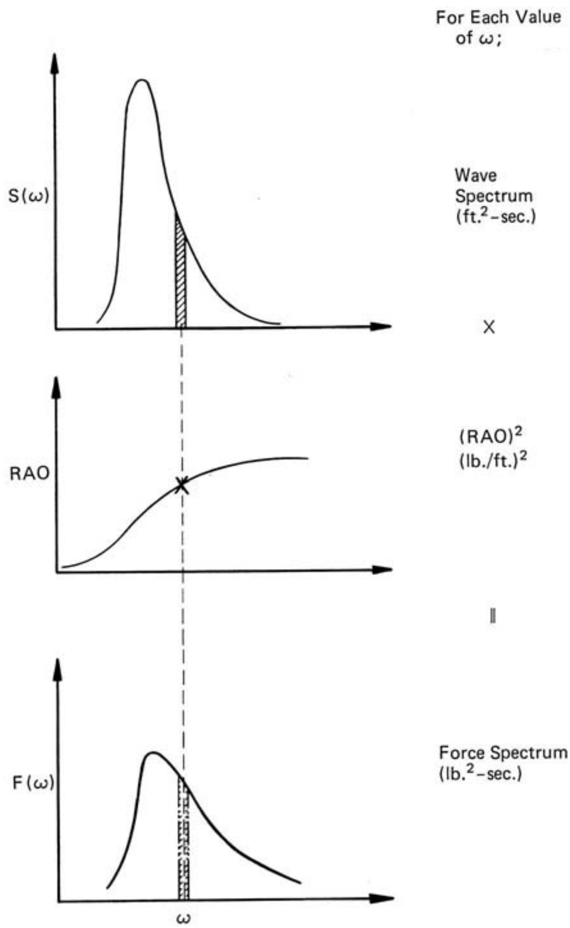

Fig. 6-22. Response spectrum analysis schematic

Important conclusions of Bomze's study for this particular case are that mooring-line forces are induced almost entirely by waves of longer than 20 s, with major contributions occurring in the period range of 40 to 200 s. Rolling is excited primarily by swells in the 13- to 19-s range. Pitching is small and insensitive to wave period, whereas heave motions tend to equal the wave height at long periods and are smaller with short-period waves. It is important to note, however, that peak dynamic loads may occur with much shorter (i.e., 6- to 10-s) wave periods than reported above, depending upon circumstances. As has been previously noted, the evaluation of a vessel's motion response to given sea conditions is important to the planning, design, and operations of a terminal facility. A vessel's response depends importantly on its orientation to the sea as well as to the wave heights and periods.

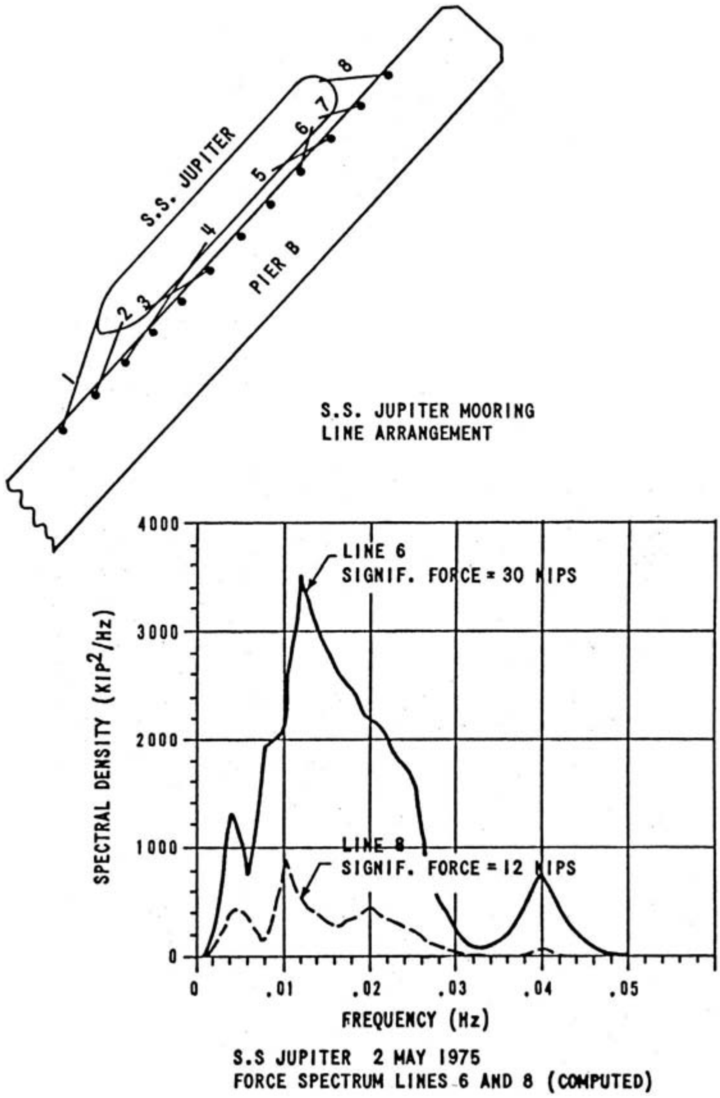

Fig. 6-23. Example of mooring line force spectra

Source: Adapted from Bomze (1980)

Fig. 6-25, after Sugin (1983), shows the relative motion response (ship motion amplitude/wave amplitude) in the six degrees of freedom for a 250,000-DWT ore carrier as a function of wave period and wave direction. Such information, developed for specific site conditions, is essential in optimizing the berth orientation. Other notable ship mooring studies of interest include Lee et al. (1975), van Oortmerssen (1976), Stammers and Wennink (1977), Khanna and Birt (1977b), Eryuzlu and

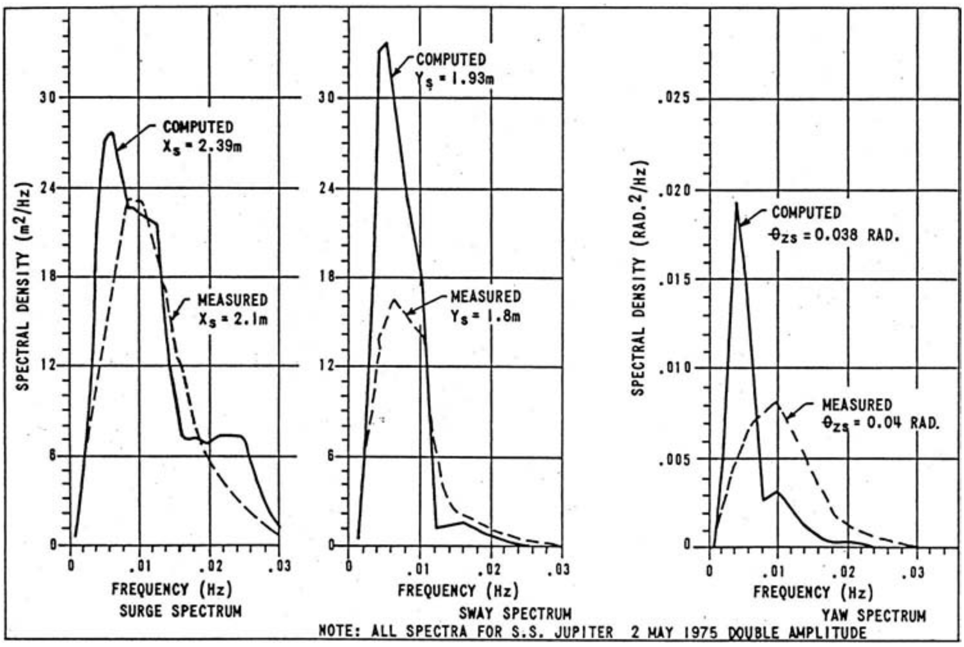

Fig. 6-24. Example of vessel motion response spectra

Source: Adapted from Bomze (1980)

Boivin (1978), Koman and Seidl (1979), Fang and Scheider (1980), Hwang and Bando (1987), van Oortmerssen et al. (1986b), van der Molen et al. (2003), and van der Molen and Ligteringen (2005). Moored vessels are most susceptible to experiencing difficulty, for example, excessive motions resulting in operational limitations and berth downtime and/or high mooring loads resulting in line breakage or other damage, under the following conditions:

- Beam-to-sea orientation,
- Long waves at resonant periods,
- Light ballast condition versus fully loaded, and
- Slack or improper mooring line tensions.

It is interesting to note that larger vessels moored alongside in harbors and subject to short-period wind waves (generally less than 3 to 4 s) and wave heights of less than 3 to 4 ft do not seem to experience any great difficulty. In general, the short pulse duration and the large vessel inertia combine to greatly reduce the local peak-wave pressure on the vessel as felt by the mooring system. The short-crestedness of the waves generally also results in peak loads that only act over a short length of vessel at any given instant.

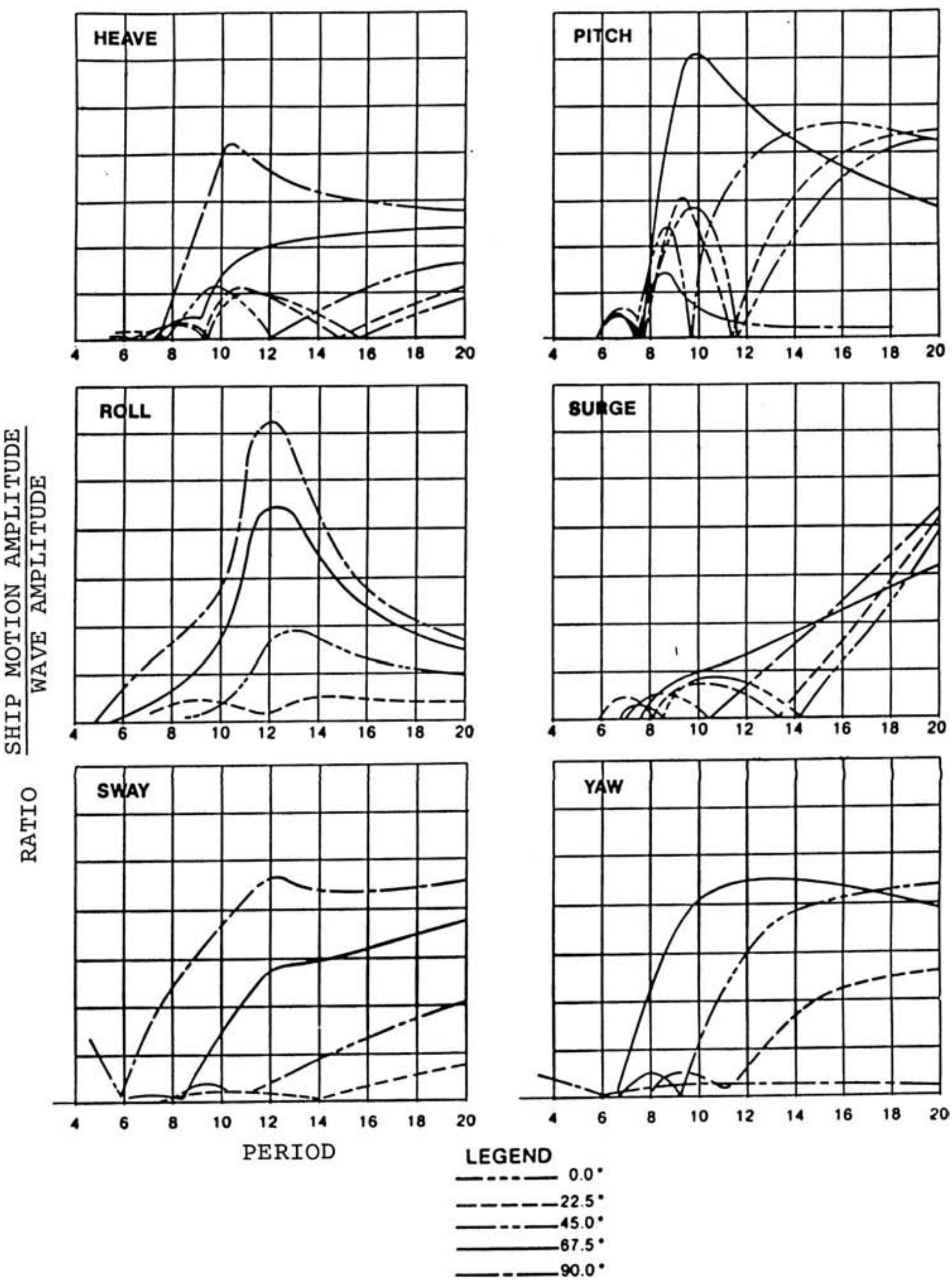

Fig. 6-25. Example of relative vessel motions as a function of wave period and direction

Source: Adapted from Sugin (1983)

6.9 Analytical Treatment and Modeling

Rigorous mathematical treatment of the dynamic analysis of combined wind, wave, and current loads, vessel motions, and mooring system response is complex. Advances in the application of high-speed computers and computational techniques have allowed the use of mathematical models in solving the general problem. Early attempts at analytical solutions to the problems of moored ships were presented by Wilson (1951, 1958, 1959). Subsequent progress in the development of the analytical treatment of moored ships can be followed in the proceedings of the NATO Advanced Studies Institute's specialty conferences on the subject (NATO 1965, 1973, 1987). Recent computer applications are described by van Oortmerssen (1976) and van Oortmerssen et al. (1986a), de Kat and Wichers (1991), Seelig and Headland (1998), and van der Molen (2003).

There are two general approaches to the mathematical simulation of ship moorings: frequency-domain analysis and time-domain analysis. Frequency-domain analysis idealizes the moored ship as a linear system characterized by a mass, spring, and dashpot, and was discussed in the previous section. The analysis is based on mechanical vibrations theory and involves solution of an equation of the following form:

$$(M + a(\omega))\ddot{x} + b(\omega)\ddot{x} + cx = F(t) \tag{6-27}$$

where

M = vessel mass,
$a(\omega)$ = added mass coefficient,
x = vessel displacement,
x = vessel velocity,
$\ddot{x}$ = vessel acceleration,
b = damping coefficient,
c = linear spring constant, and
$F(t)$ = time-varying forcing function.

In reality, there is a separate equation of the form of Eq. (6-27) for each of the six modes of vessel motion.

Terms on the left-hand side (LHS) of Eq. (6-27) account for forces imposed on the vessel as it moves in still water (i.e., in the absence of waves). The added mass, $a(\omega)$, accounts for those hydrodynamic forces proportional to acceleration of the vessel in still water. Similarly, the damping coefficient, $b(\omega)$, represents hydrodynamic forces proportional to the velocity of the ship in still water. The added mass, $a(\omega)$, and the damping coefficient, $b(\omega)$, are collectively called the hydrodynamic coefficients and are a function of the vessel motion frequency, ω. The spring constant, c, accounts for forces proportional to the vessel displacement and represents either hydrostatic restoring forces or a linearized mooring restraint, or both. $F(t)$, on the right-hand side (RHS) of Eq. (6-27), is a sinusoidally varying wave force and is a function of wave amplitude, frequency, and direction. $F(t)$ normally is computed by assuming that the vessel hull is held rigidly. Hydrostatic restoring forces are computed from basic principles of naval architecture and are discussed in Chapter 9. The hydrodynamic coefficients, $a(\omega)$ and $b(\omega)$, and the wave "forcing function," $F(t)$, usually are computed by using either slender-body or diffraction hydrodynamic theory for a variety of motion frequencies. Similarly, the wave forcing function is computed for a variety of wave frequencies and directions. Eq. (6-27) is then solved for the vessel displacement, velocity, and acceleration for each wave frequency and direction of interest. A consequence of this approach is that the vessel responds sinusoidally at a frequency equal to the wave frequency. Because an irregular wave field is characterized by a variety of frequencies and directions, Eq. (6-27) is solved for the range of wave frequencies and directions present in the wave field. Results of such analysis normally are computed for waves of unit amplitude and are summarized in terms of RAOs such as those presented in Fig. 6-25. Because the moored-ship system is linear, the motions of the vessel in an irregular wave field can be determined by using the spectral-response techniques described in the previous section.

The frequency-domain approach has been widely used because it is simpler and requires less computational effort than time-domain analysis. Its shortcomings, however, are fundamental, and frequency-domain techniques can, in many cases, provide misleading results. First of all, the mooring restraints must be linear; that is, the mooring restraint load must be a linear function of displacement. Unfortunately, mooring lines and fenders often are highly nonlinear. When the mooring restraints are nonlinear, non-negligible subharmonic motions of the vessel can occur at frequencies that differ from the forcing wave frequency (Wood 1980). Such motions are not simulated by frequency-domain analysis. Second, as has already been discussed, vessels are subject to a low-frequency, slowly varying wave-drift force. Many moored-ship systems are characterized by relatively low natural frequencies in surge, sway, and yaw. This fact, coupled with the fact that damping is small at low frequencies, makes ship moorings prone to low-frequency excitation. Soft mooring systems, such as spread moorings and single-point moorings, are particularly susceptible to slowly varying drift forces. In fact, the first-order forces at wave frequency often are neglected in the analysis of soft, single-point mooring systems.

The shortcomings of frequency-domain analysis are overcome by time-domain analysis at the expense of added computational effort. Time-domain analysis was developed by Cummins (1962) and has been described in detail by van Oortmerssen (1976). It consists of solving six simultaneous equations of the following form:

$$(M + m')\ddot{x} + \int_{-\infty}^{t} K(t-\tau)x \, d\tau + cx = F(t) \tag{6-28}$$

where

m' = constant inertial coefficient,

$K(t-T)$ = impulse-response function where T is a variable integration time denoting an earlier vessel position,
$F(t)$ = arbitrary forcing function, and
c = hydrostatic restoring force coefficient.

Other terms are as previously defined for Eq. (6-27). The terms M, m', K, and c are 6×6 matrices representing coupled motions in the six degrees of freedom: surge, sway, heave, roll, pitch, and yaw. Like the frequency-domain approach, terms on the LHS of Eq. (6-28) account for forces on the ship when moving in still water. In contrast to Eq. (6-27), however, the constant inertial coefficient, m', and the impulse–response function, $K(t-T)$, are independent of motion frequency and represent hydrodynamic forces on the vessel for any arbitrary vessel motion. The RHS of Eq. (6-28) is an arbitrary force–time function that may include first-order wave frequency forces, second-order wave drift forces, nonlinear mooring restraint forces, wind forces, current forces, and forces associated with passing ships.

The constant inertial coefficient, m', and the impulse–response coefficients cannot be determined directly. Instead, they are determined from the frequency-dependent hydrodynamic coefficients, as follows:

$$K(t) = \frac{2}{\pi} \int_0^\infty b(\omega) \cos \omega t \, d\omega \tag{6-29}$$

$$m' = a(\omega) + \frac{1}{\omega} \int_0^\infty K(t) \sin \omega t \, dt \tag{6-30}$$

van Oortmerssen (1976) provides a complete description of the numerical solution of Eq. (6-28). It suffices to say that the solution to Eq. (6-28) provides time histories of vessel motions and mooring restraint loads. Example time-domain analysis results are illustrated in Fig. 6-26 for a 100,000-DWT × 239-m LOA tanker in fully loaded condition and subject to the following environmental conditions: beam on waves with $H_s = 2.0$ m at $T = 6.9$ s, beam on swell with $H_s = 0.3$ m at $T = 12$ s, beam on wind = 15 m/s and ahead current at 1.0 m/s. Two commercially available computer programs that can be used to evaluate time-domain mooring programs are TERMSIM II, developed by the Maritime Research Institute Netherlands (MARIN), and AQWA (Atkins Quantitative Wave Analysis), developed by WS Atkins Engineering Software and currently available as ANSYS AQWA (see Appendix 3 for website addresses). Computer solutions to vessel mooring analysis in general include the following steps:

- Input wave spectrum and wind and current spectra or time series.
- Input harbor bathymetry and berth geometry and properties.
- Input vessel characteristics and hydrostatic data.

- Solve diffraction of primary wave system, wind waves, and swell.
- Solve direct response to primary waves.

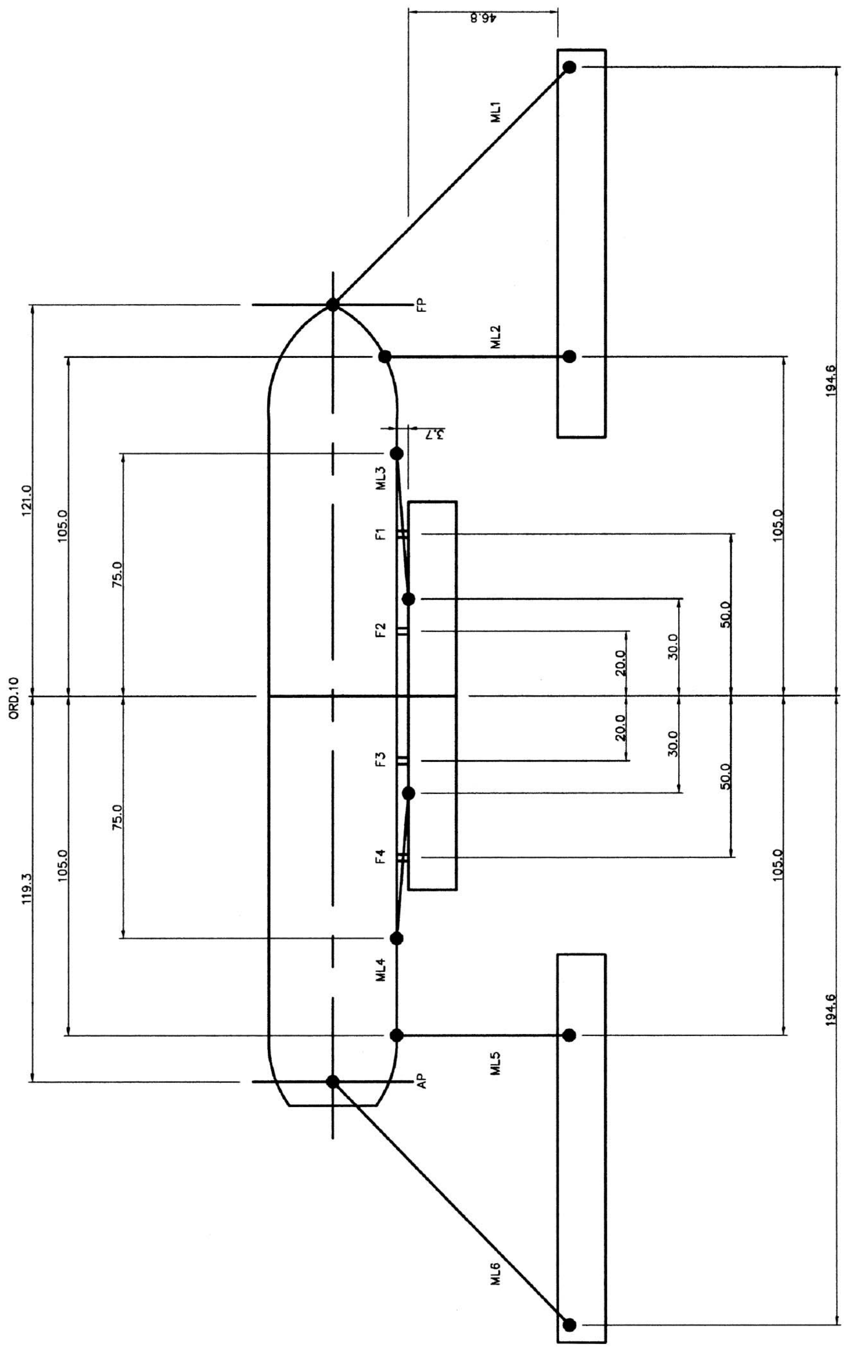

Fig. 6-26. (in 3 parts) Example time-domain analysis for a moored 100,000-DWT tanker from TERMSIM II computer program

Source: Courtesy of Moffatt Nichol Engineers

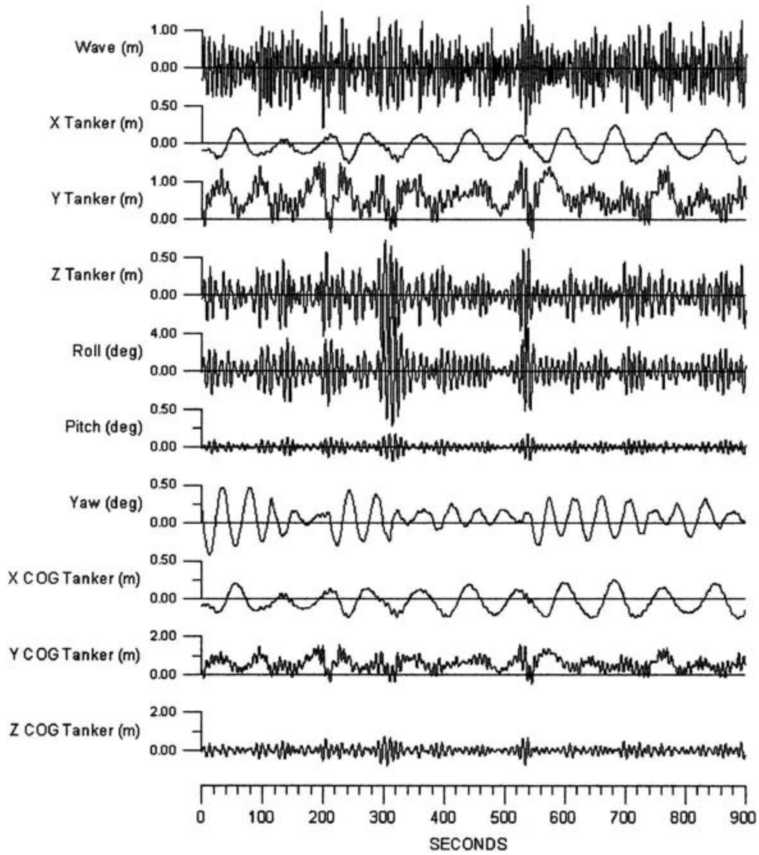

Fig. 6-26. Continued

- Solve hydrodynamic coefficients, added mass, and damping.
- Solve nonlinear response: irregular waves, long waves, and directional seas.
- Account for local effects: UKC, proximity of quay walls, and other features.
- Account for mooring lines and fender stiffness.

More in-depth treatment of the mathematical development and solutions to Eqs. (6-27) through (6-30) are beyond the scope of this text; the reader is referred to the previously cited literature for discussion of the applications of mathematical models to vessel mooring problems. General description of the use of computer simulations can be found in Bomze (1974), van Oortmerssen et al. (1986), and Hooft (1986). Analytical treatment of berthing impacts has been presented by Fontijn (1980), Headland (1992), and Lee et al. (1975), and in the various papers of the NATO conference proceedings. Leblanc et al. (1995) describe some of the limitations of time-domain dynamic analysis and provide a consistent methodology for mooring system assessment based upon quasidynamic principles.

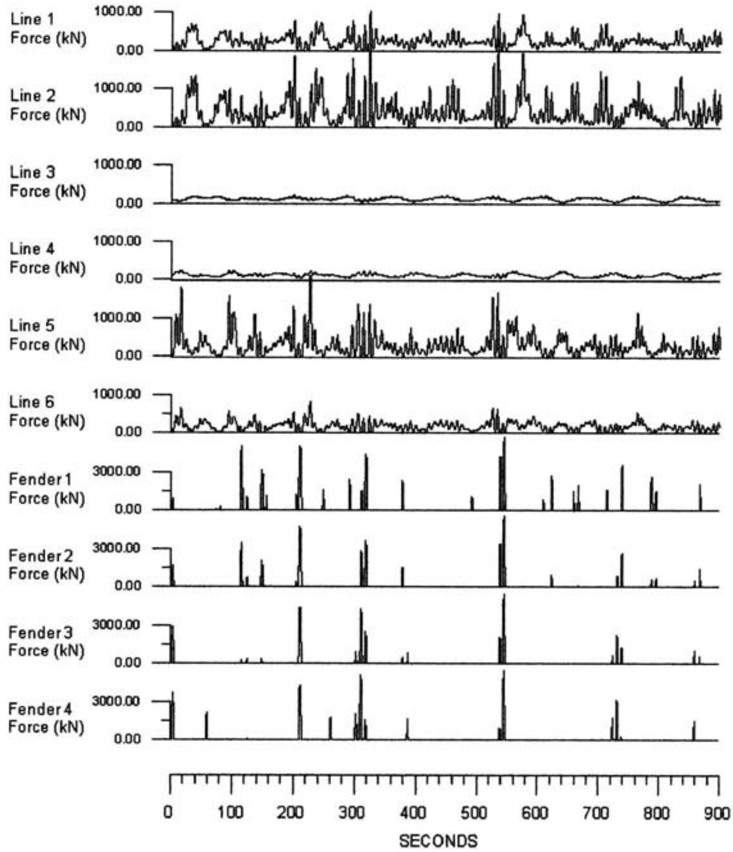

Fig. 6-26. Continued

6.10 Physical Model Testing

Physical scale modeling may be resorted to on large-scale projects of complex geometry or to validate the predictions of mathematical models. Physical hydraulic modeling is expensive and time-consuming, and thus usually used to model the final design configuration and to examine specific effects. Field studies are normally required to collect data to calibrate the model. The test setup must include the entire harbor basin or sufficient surrounding area and adjacent structures and features. Mooring line and fender stiffness must be properly accounted for. Hydraulic models are used in a wide variety of port and harbor problems, such as breakwater, wave, and water circulation studies, and to obtain mooring forces and vessel movements under variable environmental conditions. These models are based upon the principles of dimensional analysis and mechanical similitude as described in most fluid mechanics texts and such references as Newman (1977), Sarpkaya and Isaacson (1981), and Hughes (1993). The textbook by Hughes (1993) offers a thorough treatment of physical modeling and laboratory techniques for coastal engineering applications. Davies et al. (2001) present an overview of the use of physical models used in port design, and Luai and Yuanzhe (2013) describe the physical model testing of a model containership moored to a dock to determine vessel motions and mooring forces. Usually, only one or two types of force or behavior can be modeled in a given model because of relative-scale effects. Certain nondimensional numbers that express the relationship between natural forces must remain constant between the model and the prototype in order for the forces to scale properly. For example, in determining mooring forces caused by wave and current action, gravity and inertia forces govern, and their relationship is derived from the Froude model law, wherein the Froude number (N_F) must remain constant between model and prototype and is given by

$$N_F = V/\sqrt{gL} \tag{6-31}$$

where V represents the fluid velocity. Note that N_F may be based upon either the water depth or the vessel waterline length, depending upon the application. The relationships between any linear dimension (L) for a linear scale of $1:L$ and other dimensional units in a Froude model are

Acceleration, α	$\propto 1:1$
Velocity, V	$\propto L^1/2$
Time, t	$\propto L^1/2$
Force, F	$\propto L^3$
Force/unit length (or stiffness), F/L	$\propto L^2$

Model scales are usually greater than $1:100$ in order to reduce scale effects from viscous and friction forces and other factors. Fig. 6-27 shows a model test setup for obtaining the motion response and mooring forces on a 200,000-DWT prototype tanker as used by van Oortmerssen (1976) to confirm his theoretical analysis and mathematical model. The scale of the model was $1:82.5$, wave heights were measured with an electrical-resistance probe, and forces were measured by six strain-gauge-type force transducers. The results for hydrodynamic coefficients and transfer functions often are plotted against the nondimensional frequency

$$\omega' = \omega\sqrt{\mathrm{L/g}}$$

for such tests.

In accordance with the principles of dimensional analysis, the force F caused by wave action, for example, is in general a function of various other dimensions as given by

$$F = f(\rho, g, H, L, D, d)$$

In this case, D is any characteristic width dimension, and ρ is the mass density (note that the product ρg = the unit weight of fluid, γ). We can write the nondimensional forces as

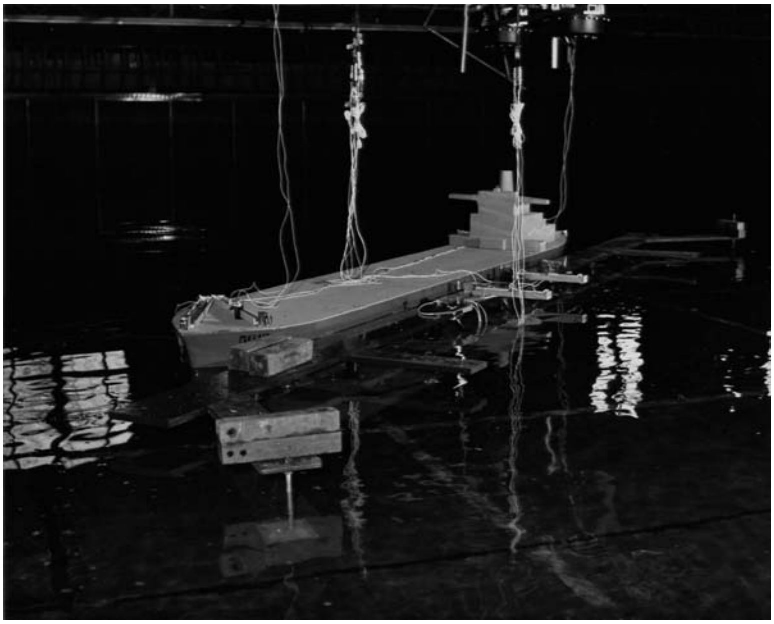

Fig. 6-27. Scale-model test setup to evaluate vessel motions and mooring forces for vessel moored alongside pier

Source: Photo courtesy of the Netherlands Ship Model Basin (NSMB)

$$\frac{F}{\rho g H D^2 = f(d/L, D/L, H/L)} \tag{6-32}$$

which is a common form of presenting wave force data.

Where viscous drag forces are important, the Reynolds number (N_R) expresses the relationship of inertia and viscous forces and is given by

$$N_R = \frac{VD}{v} \tag{6-33}$$

where ν is the kinematic viscosity, which at standard sea level pressure and temperature of 59°F (15°C) is 1.282×10^{-5} for seawater, 1.938×10^{-5} for freshwater, and 1.615×10^{-4} ft^2/s for air. In the general case of a time-invariant in-line force, such as a steady current drag force, N_R should be included as a constant parameter on the RHS in Eq. (6-25). In the linear diffraction case with $D/L > 0.2$, flow separation effects are negligible, and N_R may be neglected. Also for the linear diffraction case (see Section 4.5), the wave steepness usually is small, and the H/L term can be dropped from Eq. (6-25). Thus, the nondimensional force $F/\rho g H D^2$ can be evaluated in terms of constant d/L, D/L, or d/D values.

Kirkegaard et al. (1980) demonstrate the importance of considering directional seas in model testing. Their model test results on a fixed moored tanker subject to both two-dimensional and three-dimensional seas revealed important differences in mooring forces. For the head-sea condition, the three-dimensional representation resulted in twice the mooring force of the two-dimensional representation. In a quartering sea, there was little difference, except for greater roll motion in three dimensions. In the beam sea, the three-dimensional representation resulted in one-half the mooring forces and roll motion of the two-dimensional case. The greatest forces were generated in the beam-sea condition.

6.11 Operational Considerations

Port Operations and Use of Tugs

Port operations revolve around the berthing, mooring, loading, and unloading of vessels at berth. Normally, the ship's owners and crew are responsible for supplying and tending the mooring lines, and the port facility owners and operators are responsible for providing a safe berth with ample fenders and mooring hardware to secure the vessel and to advise the ship's crew of any potential hazards, such as strong currents or other conditions such as water depths, that they may need to be aware of. Pilots, tugs, and shoreside line handlers are responsible for the berthing of the ship until it is safely secured, and regulatory authorities, such as harbormasters and the U.S. Coast Guard, may also have some oversight in the process.

During berthing, mooring lines may be used to help slow the vessel and help guide it into position by "snubbing" against mooring hardware, possibly resulting in high bollard loads. During cargo-transfer operations, the ship may need to be repositioned along the face of the pier or even turned around to present its opposite side to the pier, a process termed "winding ship." In some instances, the corners of the pier or wharf may be used to breast against in order to rotate the ship, termed "warping." Vessel operations and movements at berth are described in detail by Clark (2009).

Tug assistance is common during berthing and may occasionally be required to hold a vessel in position or reposition it while in berth. Contemporary tugs can be quite powerful and may have sufficient power to fully compress and even damage fender systems. Forces caused by tugs may be considered in mooring analysis as a concentrated force applied at some known or assumed location on the ship's side. Many large vessels have the safe locations for tug contact painted on their sides. The maximum force exerted by a tug is known as its *bollard pull* and is related to the tug's brake horsepower (bhp) or shaft horsepower (shp) with the tug at zero speed and 100% propeller slip. An approximate rule of thumb applicable to most tug types gives the bollard pull as 10 to 15 tons per bhp. Contemporary harbor tugs in major ports typically have bollard pulls within the range of 50 to 80 tons. This range may approach or even exceed the safe working load of the ship's mooring hardware and thus puts a practical limit on tugs involved with berthing and mooring operations within relatively protected ports. Larger tugs may be required at more exposed locations. Hensen

Table 6-5. Bollard Pulls for Selected Tugboats

Tugboat	Power (hp)	Size (tons)
U.S. Navy yard tugboat (YTB)	2,000	27
Z-drive tug	3,200	45
Azipod drive tug	4,200	60
Cycloidal drive tug	4,800	50
Z-drive tug	5,100	60
Cycloidal drive tug	8,000	80
Offshore tug	15,000	200

(2003) provides thorough treatment of tug requirements and port operations. Bollard pulls for some selected representative tugboat types are listed in Table 6-5.

Many contemporary vessels, including almost all cruise ships, have thrusters typically located near the bow and stern that can propel the vessel sideways and hence usually berth without the aid of tugs. Vessels so equipped can turn around within their own length and exert good control over the berthing maneuvers. In situations with strong crosswinds or currents or at more exposed locations, however, the thrusters may lack sufficient power to overcome the environmental forces, and standby tugs may still be required.

Tide and Draft Changes

The decks of moored vessels may be subject to large changes in elevation relative to the pier deck because of changing water levels associated primarily with tides and because of changes in vessel draft associated with cargo-transfer operations. Fig. 6-28 illustrates these changes for a fully loaded ship at low water to a light/ballasted ship at high water and the associated effects on mooring line geometry. Without proper line tending, dramatic increases in mooring line tension and uplift forces on bollards

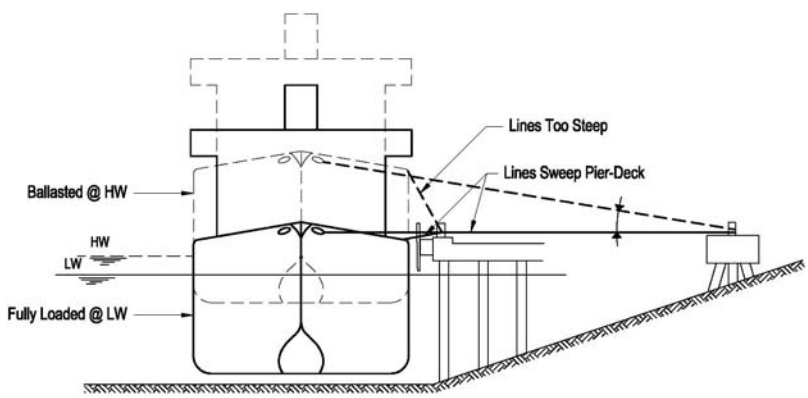

Fig. 6-28. Vessel mooring line geometry schematic

may occur with a rising tide and unloading vessel. At low water, mooring lines may chafe on the pier deck edge or foul on fenders or other appurtenances. Vessels may also trim and list during cargo-transfer operations, which may also result in changing mooring line tensions. Tankers and bulk carriers in particular undergo large changes in draft between loaded and unloaded conditions and typically take on ballast water to maintain trim and stability in light condition. The ballasted draft of tankers may be roughly estimated at around 40% of the full load draft. They also typically have considerable trim down by the stern to keep their propellers immersed.

Allowable Vessel Movements

The overall adequacy of a vessel berth and mooring system is determined not only by its structural integrity and safety under environmental conditions, but also by the limitation of vessel motions to facilitate cargo-handling operations; to prevent possible damage to the ship, pier, and/or loading equipment; and to promote safety of personnel. Extreme motions at berth are caused primarily by wave action. Table 6-6, reproduced from PIANC (1995), gives generally accepted ranges of

Table 6-6. Recommended Motion Criteria for Safe Working Conditions

Ship Type	Cargo-Handling Equipment	Surge (m)	Sway (m)	Heave (m)	Yaw (°)	Roll (°)	Pitch (°)
Fishing vessels	Elevator crane	0.15	0.15	0.4	3	3	3
	Lift-on/lift-off	1.0	1.0				
	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/Ros	Side ramp[a]	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 vessels		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[b]	3.0				
Gas tankers	Loading arms	2.0	2.0		2	2	2

Note: Motions refer to peak–peak value (except for sway, which is zero-peak).
[a]Ramps equipped with rollers.
[b]For exposed locations, 5.0 m (regular loading arms allow large movements).
Source: Data adapted from PIANC (1995).

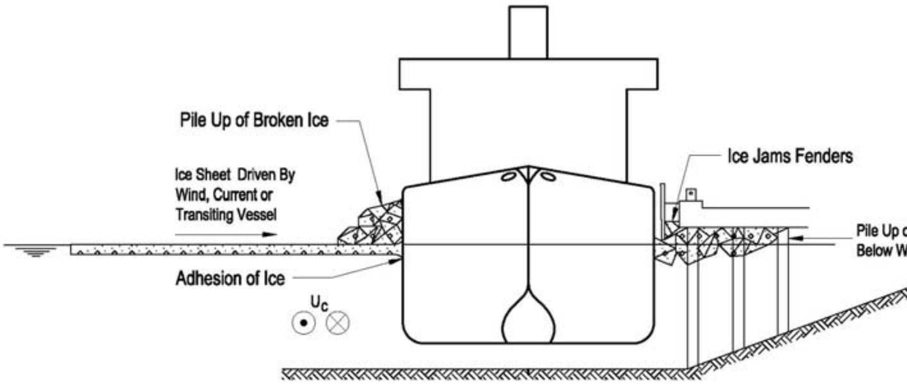

Fig. 6-29. Vessel alongside wharf in ice schematic

motion in the six degrees of freedom for various vessel types and loading operations. In-depth treatment of vessel movements and cargo-handling operations for containerships is provided by PIANC (2012c), which supplements the PIANC (1995) report. ASCE/COPRI (2014) provides a more in-depth treatment of allowable vessel movements, acceptable wave heights, and possible mitigation measures. Additional discussion of allowable vessel movements in port can be found in Thoresen (2014), Jensen et al. (1990), Bruun (1989), Slinn (1979), and Bratteland (1974).

Ice Effects

Vessels can generally be moored and operate safely in static ice of moderate thickness without incident. An important consideration is the possible horizontal thrust from moving ice and/or intact ice sheets driven by wind and current against the vessel or by adhesion to the sides of the moored vessel and the related possible pileup of ice floes against the vessel (Fig. 6-29). Forces can be potentially very high in cold regions with ice more than around 1 ft thick, especially in rivers or channels with fast currents, and ice floes, especially at time of spring breakup, when shore-fast ice breaks free of the shoreline upstream. Ice sheets may also be thrust against moored vessels by transiting vessels, which can progressively pile up ice because of refreezing after subsequent vessel passages. Other ice-related mooring issues include pileup of ice beneath a pier and jamming and/or freeze-up of fenders. Vessels operating in cold regions should be well tended and prepared to evacuate the site when extreme ice conditions are expected. See Section 4.5 for methods of calculating ice forces and further discussion of ice effects in harbors.

Maintenance

Fender systems, mooring lines, and hardware are continually exposed to deterioration, wear, and cumulative damage. Fenders and mooring lines must be inspected regularly, and lines should be provided with chafing gear, although mooring lines are usually the responsibility of the ship. Mooring line arrangements should be carefully laid out to avoid sharp turns and abrasive points of contact. Detailed discussion of mooring line inspection and maintenance can be found in Vervloesem (2009). Inspection and maintenance of mooring hardware is covered in DOD (2001), and maintenance of fender systems is addressed in Section 11.8.

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CHAPTER 7

Design of Fixed Structures

Fixed structures to which vessels are secured directly include piers, wharves, bulkheads, quays, and dolphins. Ancillary structures, which serve to access piers or provide support for cargo-handling equipment, product lines, and so on include support platforms, trestles, catwalks, and moles. Structures may be generally categorized as being of open, pile-supported construction versus closed, fill-type construction. This chapter reviews general design principles common to both types of construction and the basis of selection between them, as well as particular aspects of open, pile-supported structure design. Aspects of closed, fill-type construction are addressed in Chapter 8. Important features of piers and wharves include crane trackage, ship services and utility systems, and such details as expansion joints, deck drains, handrails, curbs, and so forth. Fender systems are discussed in Chapter 5, and mooring hardware is described in Chapter 6. It is assumed that the reader is familiar with the general principles of structural and building code requirements for bridge and building construction. The following discussion is intended to emphasize the particular requirements of marine construction as they differ from those of traditional civil engineering works.

7.1 Structural Types and Configurations

A *dock* is the most general designation for a structure or place at which a vessel can be moored. A *pier* is a dock structure that typically projects outward almost perpendicular to or at some skew angle with the shoreline. Outside of the United States, the word *jetty* is often used for a pier, whereas within the United States, jetty generally refers to a coastal engineering structure used to stabilize coastal inlets. A pier is essentially a freestanding structure, shore-connected at one end, which allows the berthing of vessels along both sides. In some instances where deep water is not available near the shore end, the pier may be constructed in a T-head or L-head configuration, or, less commonly, a U-pier configuration, with the shore-connected stem or stems serving primarily as an access road and with vessels berthed alongside the T, L, or U portion, which is almost parallel to the shoreline. In this case, the distinction between a pier and a wharf is imprecise, and the structures may also be referred to as T or L wharves. Breakwater piers provide wave protection and may allow vessel berthing only along the protected side. Rusten and Zahn (2000) describe the construction of a 1,500-ft-long breakwater pier at U.S. Naval Station Everett, in Everett, Washington, and include reviews of various breakwater options considered. Jack-up barge-type piers, which consist of bargelike hulls floated into place and jacked up on freestanding legs spudded into the bottom, may be deployed to remote sites such as the one depicted in Fig. 1-9 at Thule, Greenland. This pier consists of four 250- × 50-ft pontoon units developed by the DeLong Corporation that have been in service for more than 60 years. Recent repairs to this facility are described by Elwood and Gaythwaite (2007). Floating piers are discussed in Chapter 9.

A *wharf*, in contrast to a pier, usually is built almost parallel to and contiguous with the shoreline, so that it also performs as a soil-retaining structure. A wharf may also be referred to as a *quay* (pronounced "key"), especially when it is of solid-fill, vertical-wall construction and is long and continuous. Both piers and wharves may be of either *open* pile- or column/pile-supported construction or of *closed*, solid-fill-type construction. Fig. 7-1 shows simplified schematic cross sections of typical pier and wharf structural types.

The most common pier construction consists of a pile foundation of timber, concrete, or steel piles supporting a timber or concrete deck. The pile bents may be designed to resist lateral loads via the use of *batter* (or raker) piles, or by being rigidly cross-braced, or via cantilever frame action with moment-resistant deck connections. Batter piles that are secured to the lower level of bracing of a cross-braced pier sometimes are referred to as *spur* piles. Piles also may be driven through prefabricated tubular steel jackets, which are cross-braced and anchored by the piles. This type of construction, called *jacket/template*, is common in deepwater offshore operations. Although this type of construction is not so common for waterfront structures, Hoshiyama (2006) reports on a deepwater containership wharf built of steel jacket template construction, saving an estimated year of construction time at the Port of Nagoya, Japan, and Hebbale (2014) reports on the construction of the Canaport deepwater LNG terminal, depicted in Fig. 1-2, with steel jacket template mooring structures secured to the seabed with pin piles. Piers also may be founded on large-diameter cylinder piles, or belled cylinders may be supported by an array of piles below the mudline. Open piers also may be supported by columns or pillars bearing directly on bedrock or a very firm substratum. Lateral stability is provided by rigid cross bracing, or sometimes by vertical shear walls, called *lamella walls*, similar in nature to full-width bridge piers. Closed pier construction may be of filled *sheet pile cells*, *diaphragm cells*, *concrete caissons*, or timber or concrete *cribs*, usually with stone fill.

Wharves may be of open-pile construction with low-height retaining structures or relieving slabs along their inshore margins. *Relieving platform* construction consists of a low-level, pile-supported deck that is filled over in order to gain stability and relieve soil pressures behind the wall in weak soil conditions. A sheet pile bulkhead or cutoff wall usually is required along its inshore edge. Relieving platforms alternatively may be constructed along the inside of sheet pile bulkheads where

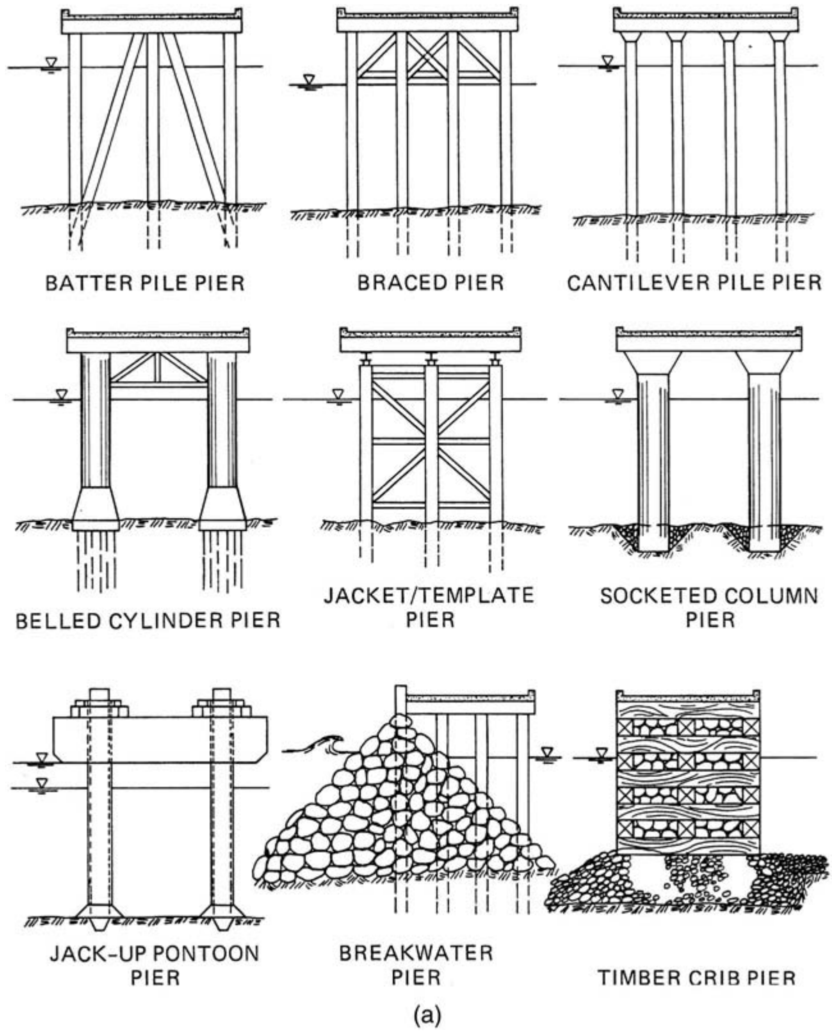


Fig. 7-1. Pier, wharf, and dolphin structural types

Source: Prepared by GZA GeoEnvironmental, Inc.

the platform support piles relieve the soil pressure surcharge on the bulkhead. The exposed bottom slope below open-type wharves usually must be protected against erosion from wave, current, and vessel wash, typically by placement of armor stone or concrete-mattress slope protection. *Bulkheads* are vertical wall structures typically constructed of steel, concrete, or timber sheet piles anchored via tie rods to discrete *deadmen* or to continuous anchor beams or walls. Cantilever bulkhead walls may be used for walls of relatively low height and low soil surcharge loadings. Cantilever walls often are made of large-diameter cylinder piles or heavy steel sheet pile Z-sections. Bulkheads also may be supported by batter piles or rock anchors.

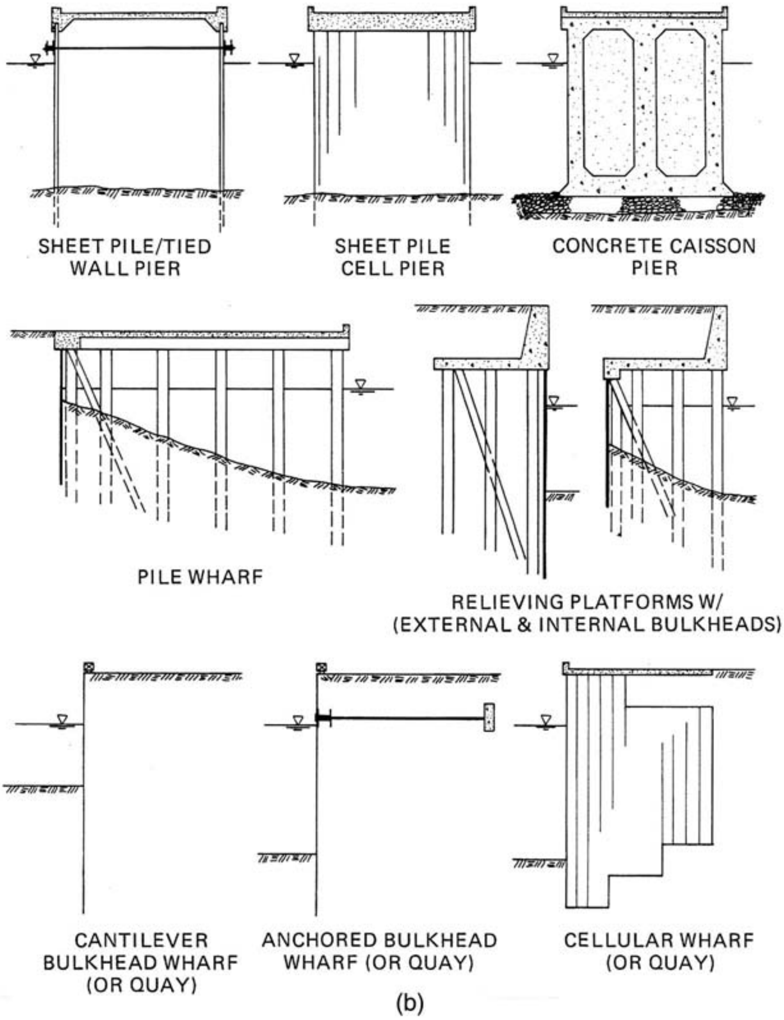


Fig. 7-1. Continued

Gravity walls gain stability against sliding and overturning via their mass, proportions, and soil friction. The simplest and earliest form of gravity wall consists of cut stone blocks, usually bedded in a foundation mat of smaller stones. Block walls also may be pile-supported by platforms, which usually are buried below the mud-line. Stone masonry may be laid "open," or the gaps between stones may be bedded with smaller crushed stone or filled with grout.

Precast concrete blocks are commonly used for contemporary block walls. Stone masonry blocks typically weigh from 1 to 5 tons, whereas solid concrete blocks often weigh from 5 to 200 tons, as limited by placing equipment. Open, cellular blocks of precast concrete that are filled with earth or stone after placement can be used to

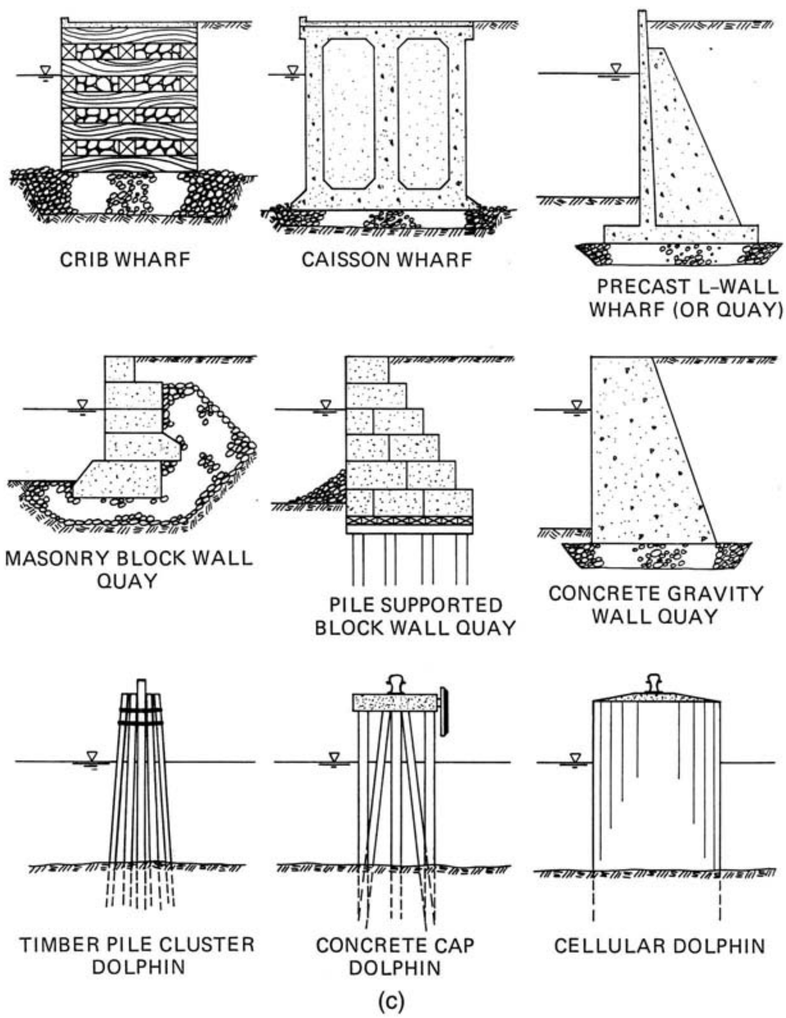


Fig. 7-1. Continued

reduce handling weights. Blocks on firm bearing material may be placed in an interlocking pattern with vertical joints staggered; where settlements are possible, they should be placed in a sloping bond pattern with the almost vertical joints aligned. Gravity walls also may be of massive "cyclopean concrete" construction poured in place, or of precast concrete caissons floated or slid into position and then filled with earth or stone. Precast *L-walls* use the weight of earth fill to gain stability against sliding and overturning. Sheet pile cells are common in wharf construction, often capped with a continuous cast-in-place concrete edge beam or deck system.

Moles and *trestles* are primarily pier or platform access structures used for vehicular, pipeline or conveyor, and/or personnel access. Moles are of solid-fill construction, and trestles are typically of freestanding pile bents or pile groups with a bridging structure spanning them.

Dolphins are solitary structures that may be used to absorb the impact of berthing ships and maintain them in position; they are referred to as berthing or breasting dolphins. They may also serve as points for securing a vessel's mooring lines, in which case they are referred to as mooring dolphins. Dolphins range from simple timber pile clusters to deepwater steel jacket structures, as described further in Section 7.4 along with special-purpose platform structures used to support cargo-handling equipment. Additional description of various pier and wharf construction types can be found in DOD (2005a), Agerschou (2004), Tsinker (2004), Thoresen (2014), and Bruun (1989).

7.2 Selection of Optimum Structure Type

Final selection of structure type and configuration for a given site is typically determined by cost considerations. Although first cost, construction cost, is often the main consideration, the present value of the project considering maintenance and income (cost versus benefit) over time or "net present value" (NPV) is perhaps a better index for cost evaluation. In turn, the most important factors in determining the most cost-effective optimum structure type at a given location generally include the subsurface soil conditions, the depth to bedrock or firm bearing material, and the water depth. These factors usually have an important effect on the relative cost of open versus closed-type construction. As a general rule, closed-type construction, such as sheet pile cells, is favored where water depths are shallow to moderate and the depth to rock is shallow. Sheet pile cells typically are between 30 and 60 ft in diameter, and it is interesting to note that the weight of steel and hence its cost per linear foot of berth face is almost independent of the cell diameter. Cells become increasingly competitive with bulkheads and other fill-type structures with increasing water depth and wall height.

Anchored bulkheads require some minimum embedment depth into competent soils and may otherwise be cost competitive at shallow water sites; these bulkheads usually are practical and economical for wall heights up to about 25 to 30 ft, although higher ones are built. At deepwater locations with soft soils extending relatively deep below the bottom, pile foundations usually are required. Although pile or open column-type construction may be used at shallow rock locations, the cost to anchor the piles or columns to the bedrock and to provide adequate horizontal stability usually exceeds that of a suitable fill-type structure. Relieving platforms, which are a combination of open and fill-type construction, may be used to provide uplift resistance and thus lateral load resistance to pile foundations that cannot develop it within the soil. A relieving platform also reduces the required bulkhead wall height, thus extending the water depth capability of the system.

Other important factors that affect the relative cost of structure types include the local cost and availability of construction materials and labor and the accessibility of construction equipment. Factors that are not related to cost but may heavily affect the choice between open and closed-type construction include the magnitude and nature of loadings; hydraulic conditions, such as wave action and currents; fire hazard and safety-related requirements; damage susceptibility and ease of repairs; construction schedule and "weather windows"; local construction practices; and environmental and regulatory concerns over water circulation and habitat loss, which almost always favor open-construction alternatives. Construction and installation of marine structures in general are addressed in Gerwick (2007). Closed, solid-fill construction generally offers greater horizontal and vertical load capacity and impact resistance than does open-pile construction. Closed-type construction typically presents a vertical to near-vertical structure face that reflects wave energy and may be a problem if the structure face is exposed to significant wave action. Possible scouring at the face of bulkheads caused by prop wash or current action may strongly influence the choice of structure type.

In the design of pile-supported structures, the choice of pile type is of obvious importance. The relative merits of the various pile types are discussed in detail in Section 7.3. Pile bent spacing and the comparison of the capacity and number of piles versus deck system framing and slab thickness is another important obvious tradeoff (Padron and Paparis 2004). In the preliminary design of deck slabs that must support both uniform and concentrated loads, it may be useful to develop equivalency relationships equating the maximum uniform and concentrated load shears and moments, as described by Junius (1983), such that slab thicknesses and unit costs can be plotted against load levels for various pile cap and/or beam spacings.

The use of precast or prefabricated structural elements often results in overall cost savings and may be essential to meet critical construction schedules. The cost of formwork, for example, is often on the order of 10% to 15% of the total construction cost for a typical pier, and additional savings in labor and potential downtime because of weather or operations make precast concrete alternatives most attractive. Availability of ready-mixed concrete may also be an important determining factor at certain sites. Tanner (1999) presents a cost comparison of cast-in-place (CIP) concrete and precast composite concrete decks used in the same project for construction of a new container wharf and found a net 23% savings for the composite deck. In the design of pile-supported deck systems, the pile bent spacing versus deck system cost must be considered. In deep water, prefabricated steel jacket/template structures or precast concrete caisson units often are mandatory to reduce construction exposure time. Precast concrete caisson or L-wall construction is more common on large-scale projects, where many repetitive units are required, and is a viable alternative where on-site construction time must be minimized and large lateral loads resisted. Caisson and L-wall construction are generally practical for wall heights up to around 60 ft.

In considering the lowest cost alternative, the initial construction cost often is the sole determining factor because average annual maintenance costs usually are relatively low, on the order of 1% or less of first cost per annum (UNCTAD 1985). Steel and open-type structures usually require more maintenance than massive concrete or stone construction, and, if all else is equal, long-term maintenance may be the determining factor. It is interesting to note, however, that marine structures may become obsolete or require upgrading within 15 to 25 years, as opposed to a usual physical structure life of 40 to 50 years or more; so annual maintenance cost comparisons may be projected over the shorter run. Selection of appropriate design "service" life for economic analysis may also affect structure-type selection (refer to the discussion of design life in Section 3.1). Important port structures typically have nominal service lives of 50 to 80 years. Lifecycle management (LCM) and whole-life costing (WLC) techniques (PIANC 1998, 2008) may be applied to a range of structure types and materials of construction (see also Section 12.5). Mathematical service life prediction models have been developed for concrete structures and typically focus on one particular mechanism of degradation. Some of these models are proprietary, and some are public domain, and their application to the design process has been addressed by Mitchell and Frohnsdorff (2004). Steel bulkheads and piles may require substantial maintenance after 15 to 20 years, compared to potentially much longer service life for concrete structures. Pavements of almost any materials usually require substantial maintenance within approximately 20 years. If a large degree of uncertainty exists over material costs and contractor efficiency with a given construction type, alternative final design plans may be prepared to generate competitive construction bids.

In the design of certain facilities, it may be useful to know and compare the incremental costs associated with greater berth depth, live-load capacity, or other variables that impose certain operational constraints. For example, although the cost of a sheet pile cell or filled caisson structure may be greater than the cost of a pile-supported pier designed to meet current live-load requirements, the greater cost of the solid construction may be considered worthwhile when compared with the cost and uncertainty of designing the pile structure option for some assumed future capacity requirement. The cost of any operational restraints imposed by live-load limits, berth depths, and so on may be compared with the incremental first cost of the facility.

7.3 Pier and Wharf Structural Design

Final design of a pier or wharf structure evolves from a preliminary design process based upon design vessel and operational requirements, which in turn dictate the water depth and site layout, the general overall dimensions, cargo-handling and deck loads, and berthing and mooring load requirements. Design criteria for deck equipment and cargoes also must include backing and turning radii, operating tolerances such as crane capacity versus reach and swing, minimum deck storage area, space for handling a vessel's lines, shore connections, services, utilities, lighting, fire protection, and so on. Proper and ample deck drainage is critical from both an operations and a maintenance viewpoint. In determining the structure's final layout and configuration, it is important to consider the possibility of future expansion or upgrading. The design of bulkheads and pile foundations alike should be adequate for any foreseeable future dredging. The selection of appropriate construction materials with regard to design life and maintenance costs also must be given careful consideration. Marine structures in general are subjected to impact and accidental overloads, more rapid rates of material deterioration, general neglect, and cumulative effects of all the foregoing. General design criteria and considerations are discussed in Chapter 3, and Chapter 4 provides a discussion of loads.

The optimization of foundation systems and structural systems often involves a design spiral process, whereby pile spacings and capacities, for example, are optimized against deck framing spans and loads. The structural design of piers and wharves generally uses the standard building codes for concrete, steel, and timber: ACI (2014a), AISC (2010), and AFPA (2012) in the United States or the equivalent European (Eurocode EN) standards in Europe, or other national standards, as applicable. Application of these codes may be supplemented or modified by additional guideline documents specific to marine structures, as introduced in this chapter. In general, the major differences between landside and waterfront construction are found in the determination of loads and related safety factors and in the selection of materials for durability. The ACI 357.3R-14 committee report (ACI 2014b) on recommended practice provides additional design guidance for concrete marine structures. As pier construction is similar in many ways to bridge construction, the standard highway bridge design practices, such as given by the American Association of State Highway and Transportation Officials (AASHTO) (2014), are often useful in pier design. Load factors and combinations for marine structures typically differ from those used in building construction. The nature of marine work necessitates that different, usually more conservative, allowable stresses; larger minimum member sizes; and special material quality requirements be applied, often involving the designer's judgment.

The Unified Facilities Criteria, UFC 4-152-01, *Design: Piers and Wharves* (DOD 2005a) is the basic design standard for U.S. Navy, Coast Guard, and other U.S. government waterfront facilities design and is also frequently referenced in the design of other nongovernment facilities, especially within North America. This document provides load case combinations for piers and wharves with moored ship(s), berthing ships, and for the vacant conditions. Load factors (LFs) for load and resistance factor design (LRFD) and for allowable stress design (ASD) are given in Tables 7-1 and 7-2, respectively. A given load case combination is the sum of the following loads multiplied by the tabulated load factor. It is therefore most important to determine which loads are likely to occur simultaneously. Also, there may be various live load conditions associated with vehicles, cranes, or other concentrated

Table 7-1. Load Combinations for LRFD Design

	1[a]	2[b]	3[c]	4[d]	5[e]	6[f]	7[g]	8[h]
Vacant								
D	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
L	0	1.61[i]	1	0	1	1	0	0
B	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
B_e	0	0	0	0	0	0	0	0
C	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
C_s	0	0	0	0	0	0	0	0
E	0	1.6	0	0	0	0	1.6	1.6
E_Q	0	0	0	0	0	1	0	1
W	0	0	0	0.8	1.6	0	1.6	0
W_s	0	0	0	0	0	0	0	0
R, S, T	0	1.2	0	0	0	0	0	0
I_{ce}	0	0.2	0	0	1	0	1	0
Berthing								
D		1.2	1.2		1.2	1.2		
L		1.61[i]	1		1	1		
B		1.2	1.2		1.2	1.2		
B_e		1.6	1		1	1		
C		1.2	1.2		1.2	1.2		
C_s		0	0		0	0		
E		1.6	0		0	0		
E_Q		0	0		0	1		
W		0	0		1.6	0		
W_s		0	0		0	0		
R, S, T		1.2	0		0	0		
I_{ce}		0.2	0		1	0		
Mooring								
D	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
L	0	1.61[i]	1	0	1	1	0	0
B	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
B_e	0	0	0	0	0	0	0	0
C	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
C_s	1.4	1.2	1.2	1.2	1.2	1.2	0.9	0.9
E	0	1.6	0	0	0	0	1.6	1.6
E_Q	0	0	0	0	0	1	0	1
W	0	0	0	0.8	1.6	0	1.6	0
W_s	0	0	0	0.8	1.6	0	1.6	0
R, S, T	0	1.2	0	0	0	0	0	0
I_{ce}	0	0.2	0	0	1	0	1	0

[a]ASCE (2003), Section 2.3.2, Eq. (1).
[b]ASCE (2003), Section 2.3.2, Eq. (2).
[c]ASCE (2003), Section 2.3.2, Eq. (3a).
[d]ASCE (2003), Section 2.3.2, Eq. (3b).
[e]ASCE (2003), Section 2.3.2, Eq. (4).
[f]ASCE (2003), Section 2.3.2, Eq. (5).
[g]ASCE (2003), Section 2.3.2, Eq. (6).
[h]ASCE (2003), Section 2.3.2, Eq. (7).
[i]11.3 for maximum outrigger float load from a truck crane.
Source: DOD (2005).

Table 7-2. Load Combinations for Allowable Stress Design (ASD)

	1[a]	2[b]	3[c]	4[d]	5[e]	6[f]	7[g]	8[h]	9[i]	10[j]
Vacant										
D	1	1	1	1	1	1	1	1	0.6	0.6
L	0	1	0	0.75	0	0	0.75	0.75	0	0
B	1	1	1	1	1	1	1	1	0.6	0.6
B_e	0	0	0	0	0	0	0	0	0	0
C	1	1	1	1	1	1	1	1	0.6	0.6
C_s	0	0	0	0	0	0	0	0	0	0
E	0	1	1	1	1	1	1	1	1	1
E_Q	0	0	0	0	0	0.7	0	0.525	0	0.7
W	0	0	0	0	1	0	0.75	0	1	0
W_s	0	0	0	0	0	0	0	0	0	0
R, S, T	0	1	0	0.75	0	0	0	0	0	0
I_{ce}	0	0.7	0.7	0	0	0	0	0	0.7	0
Berthing										
D		1		1			1	1		
L		1		0.75			0.75	0.75		
B		1		1			1	1		
B_e		1		0.75			0.75	0.75		
C		1		1			1	1		
C_s		0		0			0	0		
E		1		1			1	1		
E_Q		0		0			0	0.525		
W		0		0			0.75	0		
W_s		0		0			0	0		
R, S, T		1		0.75			0	0		
I_{ce}		0.7		0			0	0		
Mooring										
D	1	1	1	1	1	1	1	1	0.6	0.6
L	0	1	0	0.75	0	0	0.75	0.75	0	0
B	1	1	1	1	1	1	1	1	0.6	0.6
B_e	0	0	0	0	0	0	0	0	0	0
C	1	1	1	1	1	1	1	1	0.6	0.6
C_s	1	1	1	1	1	1	1	1	0.6	0.6
E	0	1	1	1	1	1	1	1	1	1
E_Q	0	0	0	0	0	0.7	0	0.525	0	0.7
W	0	0	0	0	1	0	0.75	0	1	0
W_s	0	0	0	0	1	0	0.75	0	1	0
R, S, T	0	1	0	0.75	0	0	0	0	0	0
I_{ce}	0	0.7	0.7	0	0	0	0	0	0.7	0

[a]ASCE (2003), Section 2.4.1, Eq. (1).
[b]ASCE (2003), Section 2.4.1, Eq. (2).
[c]ASCE (2003), Section 2.4.1, Eq. (3).
[d]ASCE (2003), Section 2.4.1, Eq. (4).
[e]ASCE (2003), Section 2.4.1, Eq. (5a).
[f]ASCE (2003), Section 2.4.1, Eq. (5b).
[g]ASCE (2003), Section 2.4.1, Eq. (6a).
[h]ASCE (2003), Section 2.4.1, Eq. (6b).
[i]ASCE (2003), Section 2.4.1, Eq. (7).
[j]ASCE (2003), Section 2.4.1, Eq. (8).
Source: DOD (2005).

loads that may or may not occur simultaneously with uniform loads, for example, that need to be checked as different live load cases. Refer also to Section 7.4 for seismic design load combinations for piers and wharves.

D = dead load,
L_u = uniform live load,
L_c = concentrated live load (to be applied independently of L_u),
I = impact load (applies to L_c only for moving/wheel loads as defined in DOD [2005a]),
B = bouyancy load,
B_e = berthing load,
C = current load on structure,
C_s = current load on moored ship,
E = earth pressure load,
E_Q = earthquake load,
W = wind on structure,
W_s = wind on moored ship,
R = creep/rib shortening,
S = shrinkage,
T = temperature load, and
I_{ce} = ice pressure.

Concrete and steel structures are normally designed to LRFD criteria but must also be checked for serviceability and construction loads. Service-load factors apply mainly to allowable stress design (ASD) methods often used for timber structures. Alternative live-load reduction factors may be applied as described in subsequent sections. Higher permissible stresses and lower factors of safety may be allowable under extreme environmental conditions, as described in Section 6.1. It is important to note that piers and wharves built near the shore and along urban waterfronts and those that support habitable structures and/or are open to public access in particular may also be subject to state and local building codes. Such codes often are more restrictive in terms of such as things as ultimate pile load capacities versus settlements, for example, than criteria that would apply to offshore construction. Additional information on loads and building construction requirements in general that are often referenced in waterfront construction can be found in the ASCE/SEI *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010) and the *International Building Code* (ICC 2015).

Other important codes, standards, and guideline documents that pertain directly to the general and structural design of port and harbor structures include the British Standards Institution (BSI) *Maritime Works: Code of Practice for Planning and Design of Operations* (2010) and *Maritime Works: Code of Practice for the Design of Quay Walls, Jetties and Dolphins* (2013), the German *Recommendations of the Committee for Waterfront Structures* (EAU 2004), the *Technical Standards and Commentaries for Port and Harbour Facilities in Japan* (OCADI 2009), and the Spanish (ROM 1990). Other countries may have their own codes of practice. For the design of offshore terminal structures, useful guidelines can be found in ACI (1984) and API (1993, 2014).

The preceding chapters of this book deal with the determination of design criteria and loadings and the layout of fender systems and mooring hardware. With this background information, an experienced civil engineer should be able to proceed with the design of a marine terminal structure using the basic principles of civil and structural engineering practice. The remainder of this chapter is devoted to aspects of structural design that are not so common to land-based civil engineering structures and additional design details and features peculiar to waterfront construction. All fixed structures involve some level of soil–structure interaction (SSI). Open pile-supported structures emphasize the structural engineering aspects of design, whereas solid fill-type structures emphasize the geotechnical engineering aspects. The remainder of this chapter is concerned with structural and functional design features; geotechnical aspects of fixed structure design are covered in Chapter 8. Pile-supported piers and wharves are relatively simple structures with typically repetitive framing in a series of transverse pile bents with longitudinal beams and/or continuous deck slabs with appurtenant crane trackage, utility systems, fender systems, and mooring hardware. In addition to supporting vertical uniform and concentrated loads, the deck system must also act as a horizontal diaphragm that distributes lateral and longitudinal forces to the entire pile foundation and thus ensures that the structure behaves as an integral whole.

Pile-Supported Piers and Wharves

The determination of pile axial and lateral load capacities based upon soil conditions is addressed in Section 8.7 and in the general geotechnical engineering literature. The structural design of a pile foundation system includes the selection of a pile type compatible with subsurface conditions; the determination of axial and lateral loads and associated shears, moments, and deflections; the pier deck system vertical and lateral load distribution to individual piles; the attachment of the pile head to the deck framing; and the determination of effective column length and end fixity conditions. The outboard row of piles along the berthing face of piers and wharves should be set back some minimum distance from the berthing face in order to preclude contact by ships with bulbous bows (see Section 4.8) or other protrusions and heeling of deep-draft vessels. The Port of Long Beach, California (POLB 2012), for example, requires a minimum setback of 8.5 ft from the compressed fender face for this purpose. Figs. 7-2 and 7-3 illustrate general nomenclature for representative timber and concrete pier construction.

Pile Types and Structural Design

Pile types most common to marine construction include timber, usually Douglas fir (DF), southern yellow pine (SYP), or greenheart in the United States; steel H-piles and pipe piles; and precast/prestressed concrete solid and hollow sections and large-diameter concrete cylinder piles. Fig. 7-4 illustrates these pile types and their usual

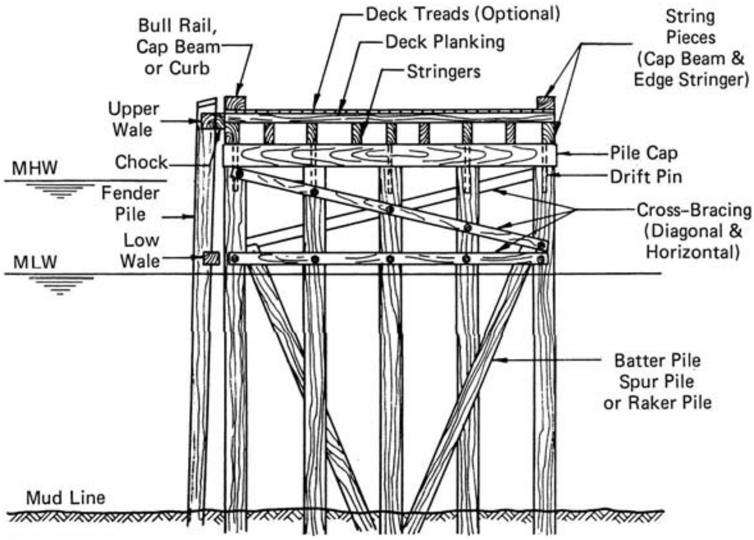


Fig. 7-2. Typical timber pier nomenclature

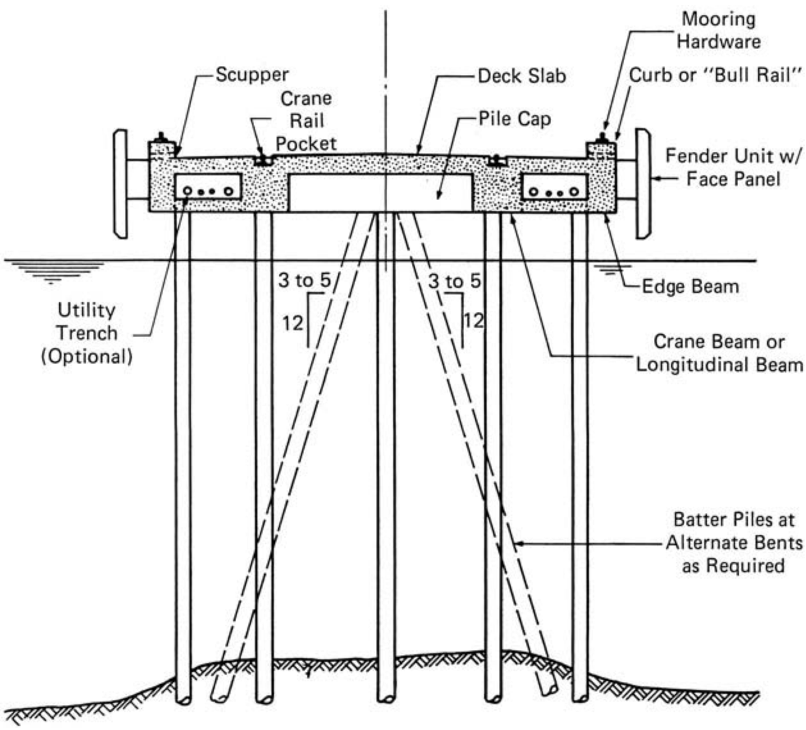


Fig. 7-3. Contemporary concrete pier nomenclature

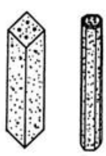
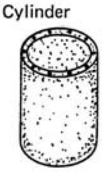
PILE TYPE	TYPICAL DIMENSIONS AND LENGTHS	USUAL RANGE OF CAPACITIES AND COLUMN LENGTHS	REMARKS
Timber 	6″–8″ Tip diameter 12″–20″ Butt diameter Douglas fir to 80′ Southern yellow pine to 65′ lengths Special order up to 125′	Typically limited to between 15 and 20 tons for all column lengths	Either Southern yellow pine or Douglas fir with ready available lengths of up to 60 feet. Piles are usually pressure treated with creosote or CCA. Greenheart piles are usually untreated.
Steel H-Pile 	Section depth 8″ to 14″ Unspliced– 60′–80′ lengths Splices– unlimited length	40 to 120 ton capacity with effective lengths of up to 60′.	Low displacement, able to penetrate through some obstructions, and easily spliced. The pile is vulnerable to corrosion and may be damaged or deflected when encountering obstructions. Favored for end bearing on rock.
Concrete Filled Steel Pipe 	8″ to 48″ ϕ 5/16″ to 3/4 wall thickness. Unspliced– 60′–80′ lengths Splices– unlimited lengths	40 to 200 tons with effective lengths up to 100′	Displacement type piles may be driven either open or close ended, easily spliced, and provides good bending resistance
Precast Concrete 	12″ to 24″ round, octagonal or square 60′ to 120′ lengths unspliced	20 to 120 tons with effective lengths up to 80′	High displacement pile may be provided with good corrosion resistance, tolerable of hard driving stresses and vulnerable to large handling stresses.
Concrete Cylinder 	30″ to 54″ diameter 150′–200′ + lengths	120 to 240 tons with effective lengths up to 250′	Prestressing allows for large handling stresses and capable of tolerating high bending stress induced by lateral loading and long unsupported lengths.

Fig. 7-4. Common marine pile types

Source: Prepared by GZA GeoEnvironmental, Inc.

dimensions and ranges of application. General criteria for pile type selection and design guidance can be found in ASCE (1996), DOD (1997), USACE (1991), and NAVFAC (1982). *General Criteria for Waterfront Construction* (DOD 2001) provides general criteria for timber, steel, and concrete piles, including a requirement that the design of all piles includes an eccentricity (e) allowance of 10% of the pile diameter times the pile load in determining the pile's capacity. Guidance for the structural design of specific pile types is reviewed in the following paragraphs.

Timber piles can be designed in accordance with the *National Design Specification for Wood Construction* (NDS) of the American Wood Council (AFPA 2012) and design guidance from the Timber Piling Council (TPC/AWPI 2002). Design stresses for DF and SYP are given by the NDS, and various adjustment factors, such as treatment, wet service, and load duration, apply. Timber piles are normally specified under ASTM D25, *Standard Specification for Round Timber Piles,* and are typically pressure-treated with chromated copper arsenate (CCA) or alkaline copper quaternary ("quat") (ACQ) (see Section 3.5) in accordance with the American Wood Protection (formerly Preservers) Association (AWPA 2015) specifications. Because timber piles are typically tapered, the structural design analysis involves the determination of the location of the "critical section" where compressive stresses are maximum with regard to the load and cross-sectional area. For a pile end bearing on rock in compression only, the critical section is at the pile tip. For a purely friction pile, the critical section is generally near and just below the mudline. Where bending stresses are involved, the critical section must be determined by more rigorous analysis or trial and error methods. Pile slenderness effects are addressed in a following subsection. Maximum loads on timber piles supporting piers and wharves have traditionally frequently been limited to around 20 tons, even though the calculated capacity may be higher.

Greenheart is a very dense tropical hardwood grown primarily in northeastern South America. It is naturally resistant to marine borers and most decay organisms and does not require treatment. Pile heads are typically banded with stainless steel straps to prevent splitting. Greenheart is more than two times stronger and stiffer than native U.S. DF and SYP piles and has more than 2.5 times the bending strength of SYP, making it an excellent choice for fender piles as well as support piles (Sproge 1958). Greenheart in the "dry," 10% to 15% moisture condition is roughly neutrally buoyant, whereas "green" wood of around 40% moisture sinks in seawater. See Section 3.5 for strength properties of greenheart.

Steel piles can be designed in accordance with the *Specification for Structural Steel Buildings* (AISC 2010). Although this standard does not specifically address pile design, it provides a relatively convenient means for calculating structural capacity of long slender columns with applied moments but assumes that columns are constructed to the specified tolerances and has no provisions for load eccentricity or accidental overloads that may not be preventable in exposed marine piles. The AISC specification is also useful for calculating the capacity of concrete-filled pipe piles, as well as unfilled pipe, and requires a 7% reduction in pipe wall thickness be applied to account for manufacturing tolerances. Concrete-filled pipe piles may alternatively be designed in accordance with ACI (2012), as described in a following paragraph. A steel pipe with a diameter to wall thickness ratio, $D/t > 35$, floats.

All steel piles should in general be designed with a corrosion allowance deducted from the new pile metal thickness and considering manufacturing tolerances, even though protective coatings and/or cathodic protection are being provided. The corrosion allowance is often on the order of around 1/16 in. in temperate climates but should be based upon expected corrosion rates over the design lifetime. Refer to Section 11.2 for corrosion rates. H-piles are especially vulnerable to corrosion because of the large surface area and two-sided exposure and also because flange edges do not hold protective coatings well. In some cases, jackets or wraps (see Sections 11.4 and 11.5) may be more cost-effective over the structure's life. H-piles are good for driving through obstructions and for end bearing on rock. Steel H-piles in the marine environment have traditionally been designed to limiting compressive stresses of 9,000 psi regardless of the calculated stress in accordance with the steel design specifications to allow for corrosion and accidental overloads.

Concrete piles are normally designed in accordance with the ACI recommended practice for concrete piles ACI 543R-12 (ACI 2012) and the ACI 318-14 (ACI 2014a) *Building Code Requirements for Structural Concrete*, which include precast, prestressed concrete, as well as reinforced concrete. Additional design guidance for prestressed concrete piles is provided by PCI (1993), which includes large-diameter hollow-core cylinder piles. ACI (2012) requires that a 5% eccentricity be applied and an additional allowable load reduction of 10% be taken for marine piles. It also has provisions for concrete-filled pipe piles and tends to be somewhat more conservative than the AISC (2010) specification, depending upon the specific design parameters. Concrete cover for precast piles in the marine environment is typically 2 in. An effective prestress of from 700 to 1,200 psi is recommended for piles longer than 40 ft (PCI 1993). A prestress of at least 1,000 psi should be considered for longer piles and where moment capacity is critical. Moment capacity can also be improved by adding plain reinforcing to the most highly stressed areas and/or to force plastic hinging (see Section 7.4) to the desired locations.

From a purely structural engineering point of view, piles of circular cross section are generally preferred because of their efficiency as long columns. They have excellent torsional strength and exhibit the same strength properties in all directions, they are easy to clean and inspect, and they generally offer less surface area for coating and corrosion than other types of equal capacity. However, cost and subsurface conditions may favor other cross sections. Steel H-piles, for example, which are often favored for end bearing on rock conditions, offer other advantages; for example, they provide better penetration through rubble and subsurface obstructions, they are relatively easy to splice and secure to the bracing and superstructure, and they usually are readily available and cost-effective. Splicing of marine piles should be avoided in general and should be restricted or prohibited within the tide zone or zones of high bending moments. Steel piles can be adequately spliced using full-penetration butt welds all around, but there is no wholly satisfactory means of splicing timber or concrete piles unless the splices are deeply buried below the point of contraflexure within the soil.

Pile Head Connections

The pile-to-cap connection is an important detail. Marine piles are often put into tension in order for the structure to resist lateral loads, and the resulting uplift forces may develop large pullout loads in the pile head. Furthermore, even relatively rigid piers and wharves with batter piles to resist lateral loads move under extreme lateral loads, resulting in moments at the pile head–cap connection. In general, minimum eccentricity factors of 10% of the pile diameter should be applied to the pile design load to give the design moment, unless pile head moments are calculated directly. Residual stresses caused by jacking piles into place during construction also should be considered. The degree of moment restraint or fixity at the pile head also affects the column length factor and the critical buckling load of the pile. Timber-pile-to-timber-cap connections and piles with shallow embedment (less than 3 to 4 in.) into concrete caps should be considered as rotation-free, pin-end-connected. The degree of fixity increases greatly with the depth of embedment into a cast-in-place concrete cap. In general, timber piles should be embedded a minimum of 6 in. into concrete caps. The use of anchor bars with or without hooks well embedded within the concrete cap also greatly increases uplift resistance (AWPI 1969). Research on steel H-piles embedded in concrete caps (AISI 1982) has shown that the tensile capacity is directly related to the depth of the anchor bars into the cap. End plates are not particularly effective in increasing either bearing capacity or pullout capacity if adequate embedment is provided. For compressive loading of steel piles, the minimum embedment should be 6 in., and the minimum edge distance from the center of the pile to the face of the caps should be from 12 to 21 in. for pile capacities of up to 110 tons and over 140 tons, respectively. If this criterion is met, then ultimate bearing stresses in the concrete of $8f'_c$ (where f'_c is the 28-day compressive strength) can be developed between the pile end area and the concrete. When supplementary pile cap reinforcement is used (circling the embedded pile), the ultimate bearing strength may be increased to approximately $10f'_c$. Larger and more heavily loaded piles require deeper pile caps and larger edge distances. Additional hoop steel reinforcement around the pile provides confinement to resist high bursting stresses created in the concrete (Mays 2015). Fig. 7-5 illustrates some representative pile-to-cap connection details. See Section 7.4 for additional seismic connection requirements and details. For all steel framing, pile tops should be welded with complete joint penetration (CJP) welds all around, preferably to a thick end plate welded to the pile cap or directly to the bottom flange of a sufficiently thick and stiffened cap beam. API (2014) provides design guidance for tubular steel connections in offshore construction.

Pile Effective Length and Depth to Fixity

The determination of the pile unsupported effective column length (l_e) and its lateral-load-resisting capability is a relatively complex process involving calculation of pile/soil relative stiffness properties with depth below the mudline under specified loading conditions. As a practical design approach, load and deflection properties of

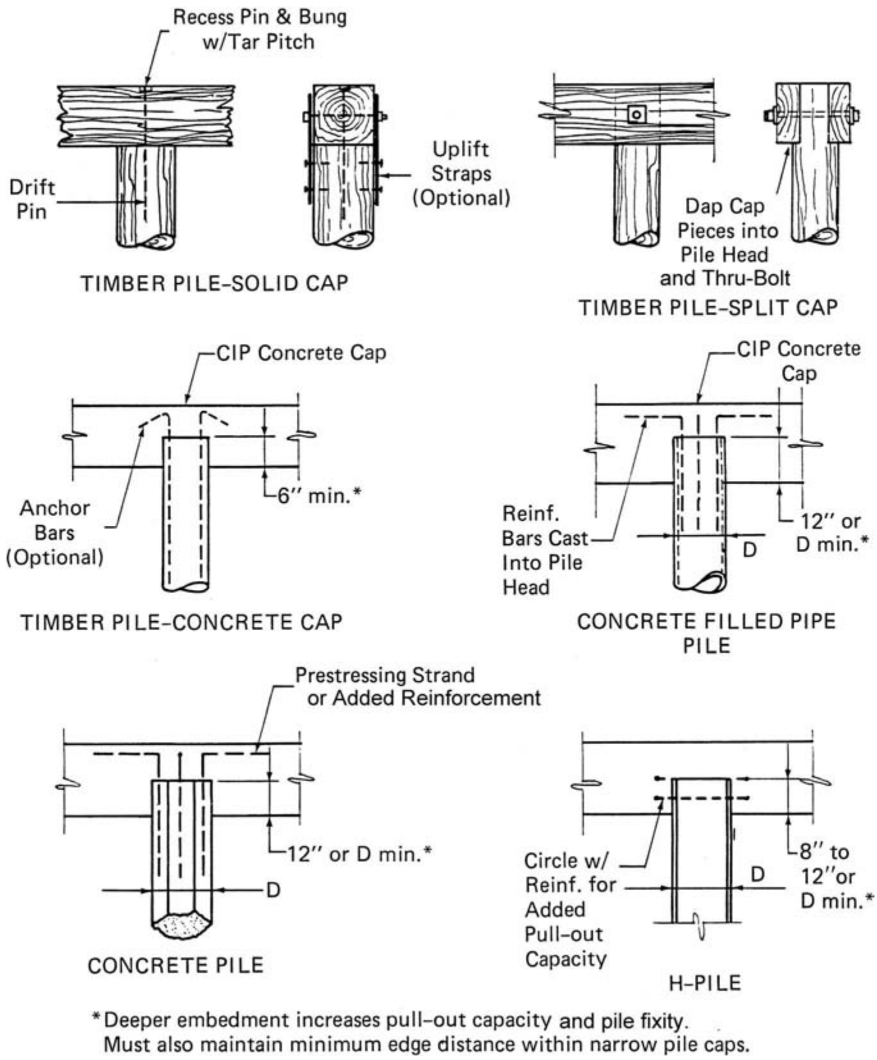


Fig. 7-5. Pile head connection details

the given soil profile with depth are plotted (p–y curves) and are related to the pile stiffness and applied load, such that an equilibrium position is found yielding shears, moments, slopes, and deflections of the pile/soil system along the length of the pile. The computer program LPILE (Reese et al. 2004) calculates pile lateral load versus deflection as well as pile/soil stiffness and p–y data for a wide range of variable subsurface conditions. The p–y method, where p is the soil resistance and y is the lateral deformation at a given depth, of calculating pile lateral capacity is described further in Section 8.7 and in greater detail by DOD (1997), Cox and McCann (1986), and Dawson (1983). NAVFAC (1982) provides nondimensional graphs to solve for the shears, moments, and deflections of laterally loaded piles of piers with both

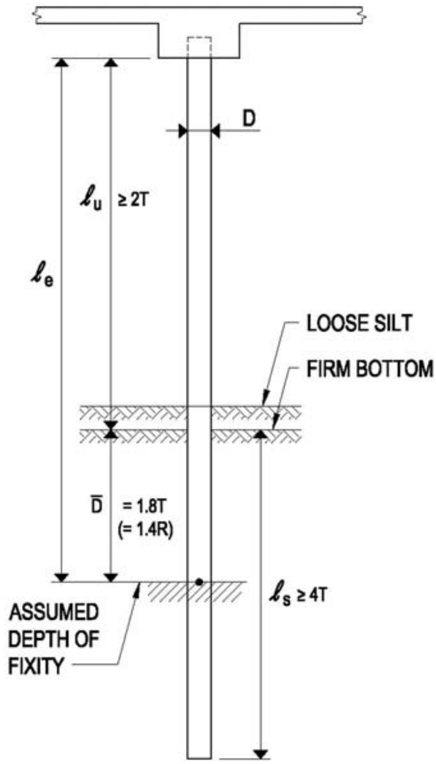


Fig. 7-6. Pile depth to fixity definition sketch

pin-connected and fixed-head conditions that can be readily applied for preliminary analysis. In the design of many nearshore and waterfront structures, however, a more simplified approach may be adequate, especially for preliminary design and checking purposes and for the calculation of pile unsupported length column analysis in particular, whereby an equivalent pile embedment or depth to fixity is assumed or calculated. The depth to fixity so found allows the pier, wharf, or individual pile to be analyzed as a simple structure in which the maximum pile unsupported length and shears, moments, and deflections at the mudline sufficiently approximate the values found in a more rigorous soil analysis. The results generally are adequate for structures in water depths of less than 75 to 100 ft. The approach assumes that the piles are sufficiently embedded in soils of uniform characteristics.

The depth to fixity ($\overline{D}$) below the mudline (refer to Fig. 7-6 for definition sketch), based upon theoretical relationships introduced in Chapter 8, can be estimated from the following equations (ACI 2012). For granular soils, silts, and normally loaded clays:

$$\overline{D} = 1.8\sqrt[5]{\frac{EI}{n_h}} \tag{7-1}$$

For preloaded clays:

$$\overline{D} = 1.4\sqrt[4]{\frac{EI}{k_s}} \tag{7-2}$$

Here, E and I are the modulus of elasticity and moment of inertia of the pile, respectively; n_h is the coefficient of the horizontal subgrade modulus; and k_s is the subgrade modulus for clay. The fifth root and fourth root terms in the above equations are known as the "characteristic length," T and R, respectively; the embedded length of the pile must be at least four times the characteristic length; and the aboveground exposed length must be at least two times the characteristic length for the above equations to strictly apply. The horizontal subgrade modulus, k_s, is assumed to be constant with depth for preloaded clays and to vary with depth for normally loaded clays and can be taken as equal to 67 times the undrained shear strength (S_u) of the soil (ACI 2012). The value of n_h for normally loaded clay is equal to k_s divided by the depth below the ground surface and can be taken as the best triangular fit of the top 10 to 15 ft on the k_s-versus-depth plot. The subgrade modulus for submerged clay is related to its undrained shear strength and consistency—soft, medium, or stiff—which is correlated with the strain at one half of the principal stress difference (ε_{50}) and is significantly reduced under cyclic loading. For stiff clays, with $S_u > 7$ psi and up to 56 psi, Reese et al. (1975) report values of n_h ranging from 500 to 2,000 lb/in.^{3} for static loading and 200 to 800 lb/in.^{3} for cyclic loading. For soft clays, the problem is more complicated, as reported by Matlock (1970), and a qualified geotechnical engineer should generally be consulted.

The value of the horizontal subgrade modulus actually varies with depth as $n_h Z = p/y$, where Z is the depth below grade and n_h is an initial value at the surface and can be used to determine the depth to fixity. There is a wide range of reported values of n_h for submerged sands ranging from approximately 2.6 lb/in.^{3} loose to 51 lb/in.^{3} dense, as determined by Terzaghi (1955), to from 20 lb/in.^{3} loose to 125 lb/in.^{3} dense, as reported by Reese et al. (1974). The results of Terzaghi can be considered generally conservative for depth to fixity calculations, as described herein. The results of Reese et al. and values given in API (2014) are recommended. Values for n_h for sands apply to both static and cyclic loading. Values of subgrade modulus and soil properties in general should be determined by a qualified geotechnical engineer for the site-specific conditions. These values may need to be modified for other effects as well, including pile spacing and group action effects [refer to ASCE (1993) and the relevant geotechnical literature]. Repeated cyclic loading in one direction reduces the subgrade modulus over time and may eventually cause a pile to permanently lean in that direction.

For piles that "fetch up" in end bearing on rock or another hard stratum above a level of approximately $2.2\overline{D}$, the pile should be considered as pin-ended at its tip. One should measure $\overline{D}$ from the top of the competent soil stratum, discounting any loose, recently deposited organic material or debris. Where the bottom slopes steeply

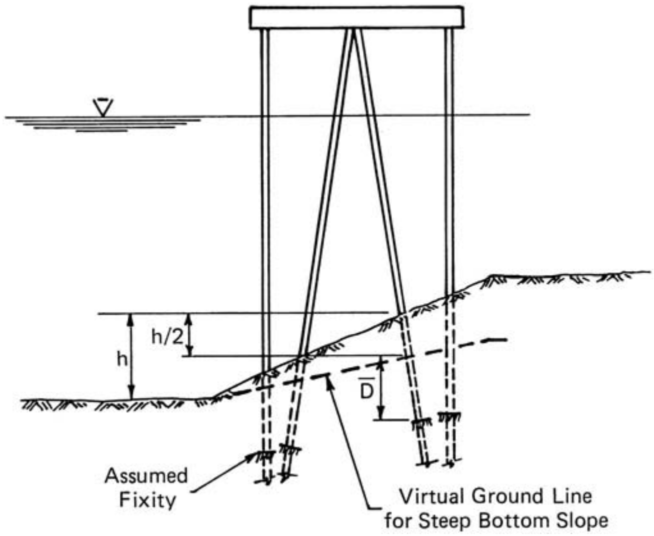


Fig. 7-7. Pile fixity for sloping bottom and batter piles

below the structure, $\overline{D}$ should be measured downward from a virtual ground line defined by a line drawn halfway between the soil surface at the given pile and the bottom (or dredge depth) at the face of the structure (Fig. 7-7). For batter piles, $\overline{D}$ should be measured vertically downward, not along the pile length. The value of $\overline{D}$ usually lies within the range of 3.5 (stiff clay) and 8.5 pile diameters (soft silt). For typical nearshore waterfront structures of interest, this is usually on the order of 5 to 15 ft. In figuring the pile unsupported length (l_e), consideration must be given to future dredging or the removal of material by scour or prop wash.

Pile Slenderness Effects

Piles supporting many marine and most port structures can be considered slender columns that have relatively long unsupported lengths with comparatively small cross-sectional areas. Such piles may be subject to failure by buckling because of elastic instability under some threshold critical buckling load (P_{cr}), as given by the Euler formula. The theory of elastic stability is well covered in the structural engineering literature. For a simple column subject to axial load and pin connected at each end, the Euler formula is

$$P_{cr} = \pi^2 EI / l_u^2 \tag{7-3}$$

which can be rearranged in terms of the sectional area, A, and radius of gyration, r, to give a critical buckling stress (σ_{cr});

$$\sigma_{cr} = \frac{P_{cr}}{A} = \pi^2 \frac{E}{\left(\frac{K_c l_e}{r}\right)^2} \tag{7-4}$$

Table 7-3. Theoretical and Recommended Column Length Factors (K_c) for Various End Conditions

End Condition	Nontranslating Head Condition	Translating Head Condition
Both ends fully fixed	0.5/0.65	>1.0/1.2
One end fixed/one hinged	0.7/0.8	>2.0/2.1
Both ends hinged	1.0/1.0	(unstable)

where $K_c l_e/r$ is known as the slenderness ratio, K_c is an effective column length factor that accounts for the pile's end-fixity conditions, and r is the least radius of gyration of the pile cross section. In general, it is desirable that $K_c l_e/r$ values be kept below 120 and preferably below 90 to 100, although much higher values may be found in deepwater piers and wharves. For values of $K_c l_e/r$ within the intermediate column range of 30 to 100, the pile behavior is relatively insensitive to variations in K_c. For $K_c l_e/r$ greater than 100, such long columns become very sensitive to K_c; and for short columns with $K_c l_e/r$ less than approximately 30, the column strength reduction factor is relatively small. For such short columns, the elastic limit may be reached before buckling, and the critical stress for inelastic buckling is determined by replacing E with a tangent modulus (E_t). Column curves can then be plotted for the unit stress for a given material versus the slenderness ratio.

Theoretical and recommended column length factors (K_c) for various end conditions are given in the ACI (2014a) and AISC (2010) codes, and other structural engineering references and are summarized in Table 7-3 for conditions of interest herein.

The values of K_c for nontranslating head conditions apply to structures that are rigidly braced or supported against horizontal forces by batter piles or rigid shore connection, and thus are essentially translation fixed. Structures supported on unbraced cantilevered piles should be considered translation-free, and the appropriate value should be confirmed by analysis.

P-Δ effects must also be considered, as described in a following discussion of dynamic response. For piers and wharves in particular where there are many piles of varying lengths, all piles are not near buckling at the same time and may hence provide some degree of translational restraint. Note that for one end fixed in the soil and the other rotation-fixed at the pile cap but subject to translation, the theoretical K_c value equals 1.0, as for two pin ends. However, a minimum value of 1.2 is recommended for "flexible" cantilever piers and structures meeting this criterion.

Considering that full fixity at cap and soil cannot necessarily be guaranteed in most marine structures, a minimum design value of K_c equal to 0.75 is often recommended for the translation-fixed case, even though fixed or fixed end conditions may be assumed. For piles that cannot be considered fully fixed below ground, some support against buckling is provided even by relatively soft soils. Appropriate values of K_c may be found by rational analysis and may be applied to the total pile length. Solutions for P_{cr} for the general case with intermediate fixity (rotational springs) and translational support (shear springs) at one or both ends can be found in Pilkey (1994). Closed end solutions for K_c for any type of framing have been put forth by Aristizabal-Ochoa (1994). Pile lateral load stiffness may need to be reduced depending upon soil conditions for spacings less than around 8D to a group reduction factor of 3.0 at 3D center to center spacing. Piles that are fitted with well-bonded structural jackets and/or pipe piles with varying wall thickness or other change in cross section may be treated as the "stepped columns" for which Agrawal and Stafiej (1980) present tabulated solutions for various end conditions, relative stiffnesses, and lengths. This type of analysis may be helpful in regaining some capacity of piles retrofitted with jackets by reducing the effective value of K_c based upon the original pile section.

Batter Piles

Batter piles are used to resist lateral loads and movements. When batter piles are used, the lateral load capacity of the vertical piles in the group is often neglected. A batter pile in compression exerts an upward force that must be resisted by the dead load of the structure or by tension in other piles in order to resist lateral loads. The pile uplift component may exert very high stresses, punching shear, in the deck framing, with high lateral loads especially during extreme seismic events (refer to Section 7.4). Pile batters usually range from 1 horizontal to 12 vertical (1:12) to 5:12, which is usually considered the flattest batter that can be practically driven without special driving equipment. It usually is assumed that a batter pile has the same axial load capacity that a similar vertical pile has when driven to the same depth within a given stratum. The use of batter piles generally results in a dramatic reduction in pier deflections. Even a relatively shallow batter of 1:12 may reduce overall deflections some 40% or more over only vertical piles.

Pile Load Distribution

Analysis of the distribution of both vertical and horizontal loads to individual piles often is simplified by reducing the general three-dimensional problem to the analysis of planar pile bents. This approach is generally valid because pier and wharf structures usually consist of a great number of uniformly spaced pile bents. Longitudinal forces acting normal to the pile bent then are dealt with separately by providing independent batter pile groups, or, in the case of a pier on cantilever piles, by designing piles for biaxial shear and bending. Vertical loads usually are reduced to bent loads by assuming a tributary area equal to the bent spacing for uniformly distributed live loads. Live-load reduction factors on the order of 25% sometimes are applied to the total uniform deck load in determining individual pile loads, depending upon the probability of the entire deck area being loaded to its design capacity at once. The distribution of concentrated loads is carried out by elastic

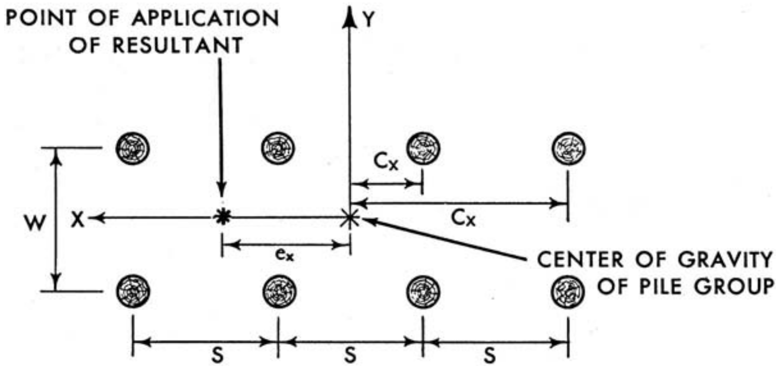


Fig. 7-8. Pile load distribution definition sketch

analysis, according to the nature of the loading. Wheel loads, for example, should be placed directly over a given pile to obtain maximum individual pile loads. It may be conservatively assumed that the entire wheel load is taken by a single pile, although a more rigorous elastic analysis usually demonstrates some degree of distribution along the pile cap, depending on its depth and rigidity. Impact factors often are neglected in calculating maximum pile loads caused by vehicles and moving equipment. Group action effects within the soil need to be checked for closely spaced piles. Pile group behavior is treated in depth in USACE (1983).

For the general case of a stationary concentrated load (F_v), the vertical load on an individual pile (P_v) may be calculated from

$$P_v = \frac{\sum F_v}{n} + \frac{\sum F_v e_x c_x}{I_x} \tag{7-5}$$

where

n = number of piles in the bent,

e_x = eccentricity of the concentrated load measured from the center of gravity (c.g.) of the pile group,

c_x = distance of the individual pile from the c.g. of the group, and

I_x = moment of inertia of the pile group about its c.g. (refer to Fig. 7-8).

The three-dimensional case can be accommodated by adding a third term to Eq. (7-3) for e_y, c_y, and I_y. The above method assumes a rigid pile cap and deck system, and that all piles have the same elastic properties.

According to the theory of continuous beams on elastic supports, a "short beam" can be considered as rigid when its length is equal to or less than about $5/\beta$, where β is the relative stiffness factor, given by

$$\beta = \sqrt[4]{\frac{K_e}{4EI}} \tag{7-6}$$

where

K_e = elastic support constant, which for a series of equally spaced elastic supports is equal to the support spring constant divided by the spacing, and

E and I = beam modulus of elasticity and moment of inertia, respectively.

Eq. (7-4) also assumes that the piles are linearly elastic in both directions, which is not strictly true for heavily loaded friction piles. Beams can be considered short when their length is less than approximately 5/β. For rigid beams, the load distribution to the piles can be treated as a statically determinate problem with a linear load distribution as reflected in Eq. (7-3); otherwise, a more rigorous elastic analysis must be performed.

When batter piles are used, the load distribution problem becomes largely indeterminate, as the lateral reactions must be balanced. When counteropposing batter piles are placed in A-frame fashion, the vertical forces cancel (provided that the uplift piles can develop adequate tension) and the analysis can proceed. Where batter piles are used only in compression and depend upon the weight of the deck system or vertical piles in tension for uplift resistance, a more rigorous analysis is required. More accurate plane-frame and three-dimensional analysis of pile bents can be readily carried out in one of the many structural analysis programs available.

Pile Bent Configurations and Lateral Load Distribution

Fig. 7-9 illustrates cantilever, braced, and batter pile bents and their lateral load resisting systems, idealized by using the equivalent pile-length/depth-to-fixity method. Note that where batter piles are used, the depth-to-fixity method cannot be used to accurately predict deflections and lateral load/deflection behavior because the structure's lateral deflection depends primarily upon the axial deflection of the pile. Therefore, the pile's elastic shortening and movement in the soil must be considered over the entire length of the pile. Pile axial stiffness is, in general, different for tension and compression. Axial load/deflection properties of the pile can be presented in the form of t–z curves, similar to the p–y curves described earlier for pile lateral deflections. Structural analysis then can proceed by replacing the portion of the pile below the mudline with an equivalent dummy pile that has soil-spring reaction forces and moments, as described in Section 8.7. For the general case of a pile group of arbitrary geometry and loadings, the computer program CPGA (Hartman et al. 1989), developed by the USACE Waterways Experiment Station (WES), is helpful.

Wharf structures may gain lateral stability by rigid connection to retaining structures or anchoring structures, such as friction slabs that are continuous along the wharf's inshore margin (Fig. 7-10). Alternatively, they may be shore connected at discrete anchor points, an arrangement that requires careful analysis of the horizontal load distribution in both directions, or they may be constructed independent of their shoreside boundary and may gain lateral stability via any of the methods shown in Fig. 7-10 for pier construction. A critical aspect of wharf design is the

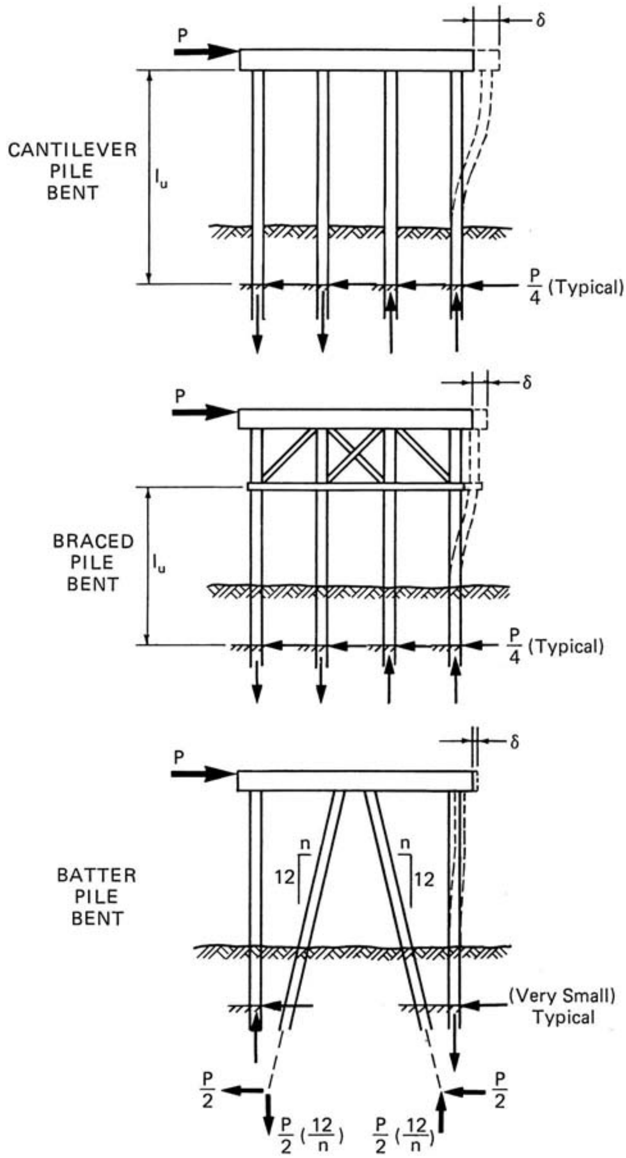


Fig. 7-9. Pile bent configurations for resisting lateral loads

analysis of backland soil pressures and overall slope stability both behind and below the wharf, as described in Sections 8.4 and 8.5. The under-deck slope of pile-supported wharves is often constructed as a retention dike for fill materials, such as from dredging of the berth, and is usually stabilized and protected from scour (see Section 4.8) with stone armor layers over filter layers. Geotextile filter fabrics are often placed between the fill and stone filter layers.

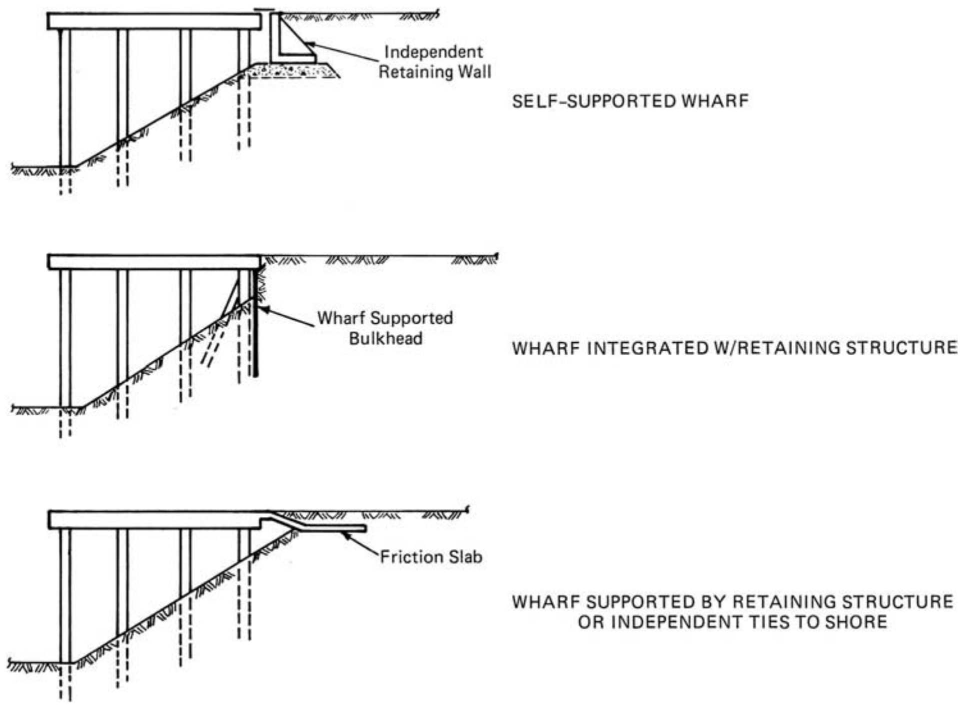


Fig. 7-10. Wharf lateral stability configurations

Lateral loads may be considered to be distributed uniformly along the pier face as a presumptive uniform load (see Sections 4.6 and 6.1) or as discrete point loads at mooring hardware or fender locations. Fig. 7-11 illustrates traditional assumptions regarding lateral load distribution according to traditional U.S. Navy guidance (NAVFAC 1980). In reality, pier deck systems typically are very rigid and, because of significant diaphragm action, even point loads concentrated directly on a single bent are well distributed throughout the structure. The pier deck can be treated as a beam on an elastic foundation, as described in the previous discussion of pile caps, with the pile bent lateral stiffness acting as elastic spring supports. In certain situations, better distribution can be obtained through the use of more flexible pile bents. Padron and White (1983) and Padron and Elzoghby (1986) applied computer methods and presented findings for example piers.

The percentage of the total vessel impact or other concentrated load taken by a given bent depends on the number and spacing of batter piles and batter pile bents, the pile batter and stiffness, the deck stiffness and the ratio of bent spacing to deck width, and the location of the impact along the pier length. In general, end bents are subject to relatively large shares of impact and mooring point loads. The simplest conservative approach is to assume that a given pile bent takes 100% of the point load. The assumption of a 45° distribution about an

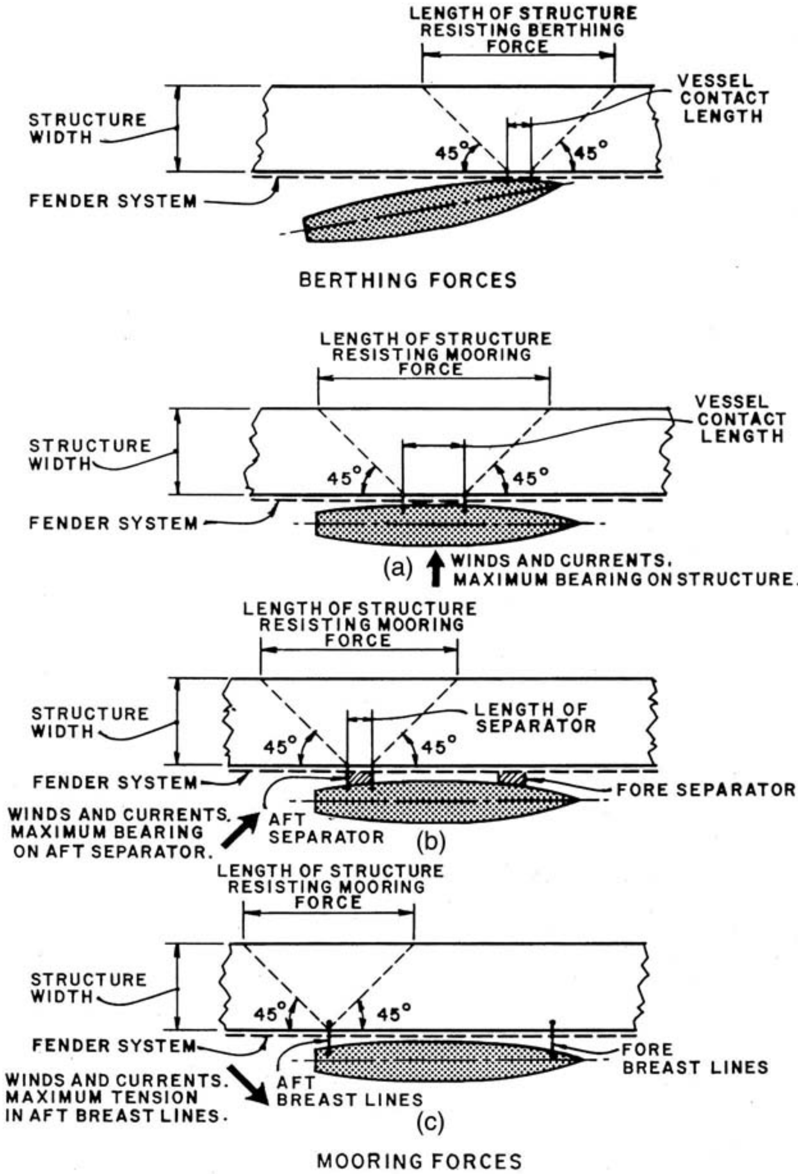


Fig. 7-11. Lateral distribution of berthing and mooring forces

Source: NAVFAC (1980)

affected bent or contact area may result in an approximately 50% to 67% reduction in this load, whereas the elastic beam analysis often results in a 67% to 90% reduction of the local impact load. The elastic beam analysis may often lead the designer to the use of more flexible bents in order to obtain a favorable load distribution and reduction in batter pile requirements. In this case, it is important also to consider the pile and overall pier deflections and dynamic response and possible vibration problems.

The placement of batter piles or other means to resist longitudinal forces should be given careful consideration. Such piles may be subject to large additional forces because of overall torque moments generated by lateral forces. Added loads in longitudinal piles caused by overall pier torque moments can be greatly diminished by the judicious placement of lateral pile bents near the ends of the pier. In general, if longitudinal forces can be taken out at a single location or at a minimal number of locations, the deck can be constructed without expansion joints over long distances because of the great flexibility of the lateral bents normal to their plane. Piers and wharves have been constructed up to 800 ft and longer without expansion joints on the basis of this principle. Ultimately, the optimum overall layout represents a compromise with the deck system design; the designer's goal is to balance all the external forces with an efficient, well-proportioned structure.

At offshore locations and at those where water depths exceed approximately 50 to 75 ft and/or those where large cyclic wave loadings govern the design, the equivalent pile-depth-to-fixity method is generally inadequate for design. At offshore locations, the pile–soil interaction must be calculated much more accurately than this method allows. Large-diameter steel pipe pile towers often are used, similar to offshore oil platform construction. The towers usually are braced and the piles slightly battered, usually on the order of 1:12 to 1:8. The design of offshore structures is treated in textbook manner by McClelland and Reifel (1986), Hsu (1984), Dawson (1983), and Graff (1981). For deepwater steel-pipe-pile construction and for structures with tubular joints, design guidance can be found in the American Petroleum Institute, RP-2A (API 2014). Offshore terminal structures also may be constructed of precast/prestressed concrete piles, with large-diameter cylinder piles favored in deep water.

Open pier and wharf construction also may consist of columns or pillars that bear directly on bedrock at sites where there is little or no soil overburden. The column bases must be socketed or doweled into the bedrock, and lateral stability is provided by batter piles or rigid bracing, similar to driven pile piers or, alternatively for wharf construction, by horizontal ties to shore anchorages. Lamellar cross walls may also be placed at strategic locations to provide lateral stability in some instances. The columns often consist of concrete-filled steel pipe sections or precast concrete. Alternatively, the columns can be of poured-in-place concrete using underwater tremie concrete techniques. Rigid quality control of the tremie concrete mix and placement is critical to the success of this method. The use of tremie concrete in column-type construction has been covered in some detail by Thoresen (2014).

Pile load tests for both lateral load and bearing capacity often are called for, especially in areas of complex subsurface conditions and where soil exploration data are minimal.

Deck Systems

Pier and wharf deck systems generally fall into one of the following broad categories: all-timber, concrete, and composite systems. Composite systems usually consist of a concrete deck slab supported on either timber or steel pile caps and stringers. All-steel deck systems are rare except perhaps on isolated offshore platform structures and on floating structures. Steel decks may be constructed of heavy-duty bridge grating. Traditional all-timber pier construction has been used for waterfront structures of all sizes, including deck loadings of up to 800 lb/ft^2 or more and railroad and heavy crane rail loadings in the past, but today the vast majority of piers and wharves for oceangoing vessels are of concrete and/or concrete composite deck construction. Some heavy timber piers built in the post–WW II era are still in operation, although many have had their decks replaced with concrete, and so it remains important for port engineers to have some familiarity with types of construction.

Timber Pier Construction

Today, all-timber pier construction usually is relegated to small-craft harbors, public facilities, and other more lightly loaded sites, where it often is the least expensive alternative. Typical timber pier construction usually consists of pile caps on cross-braced bents supporting longitudinal stringers to which the decking is secured (Fig. 7-2). Pile bent spacing in timber piers usually is within the range of 5 to 15 ft, depending upon the nature of the loadings. Pile caps usually consist of a single large timber placed across the pile butts and "drifted" into the piles with steel dowels called drift pins, or of split-cap construction, in which two timbers are dapped into opposite sides of the pile head and through-bolted. Drift pins should be recessed, and the holes should be filled with tar pitch. In the design of pile caps spanning multiple piles, it is often assumed that one or more piles are defective within a bent.

Fig. 7-12 illustrates some traditional heavy timber pier construction details. Single solid caps sometimes are additionally secured with uplift straps to resist uplift forces associated with wave and ice action and to resist lateral loads. Stringers likewise are drifted into the pile caps and also may need uplift connections and sometimes lateral bridging as required by timber design specifications (AWC 2012). Deck planks should be of quarter-sawn lumber and should be spiked into the stringers. Decking intended to support truckloads should be a minimum of 3 in. thick. Adequate gaps should be left between planks (usually 3/8 to 1/2 in.) to allow for drainage and some reduction of peak wave uplift pressures. Fastening hardware is generally of mild steel and should be hot-dipped galvanized in accordance with the appropriate ASTM specifications (see Section 11.5). Within and below the splash zone, bolts should be a minimum of 1 in. in diameter with plates of 1/2-in. minimum thickness. Above the splash zone, minimum sizes should be 3/4-in. diameter for bolts and 3/8-in. thickness for plates. "Ogee"-type washers are preferred on all connections below the deck level for their ability to distribute loads and their long-term corrosion allowance. Connection details are of utmost importance in timber design

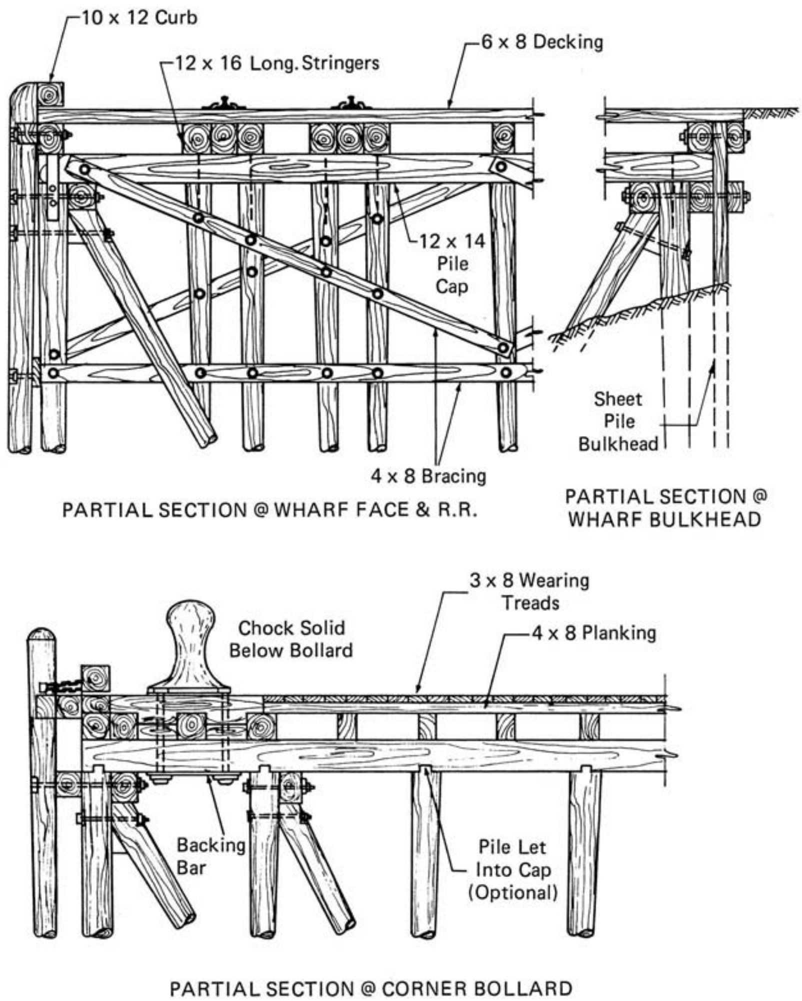


Fig. 7-12. Timber pier construction details

because it is often difficult to develop the full strength of the members being connected with the simple bolted connections often seen in waterfront construction.

Allowable stresses in timber should be reduced for wet-service conditions, although an increase in allowable stress applies to loads of short duration in accordance with the timber design specifications. Wood is excellent at absorbing impact loads, and the allowable stress can be increased by a factor of two for impact according to the NDS (AFPA 2012). However, there is evidence that the CCA pressure treatment process may make timber more brittle. Most domestic wood used in marine construction must be protected from decay and marine borer attack (see Sections 3.5 and 11.2) with appropriate preservative treatment. The use of tropical hardwoods (see Section 3.5) avoids the treatment problem and generally results in smaller member dimensions that help to offset the wood's typically higher cost. Allowable stresses for exotic timbers usually are available through timber suppliers. Additional design guidance for timber structure design can be found in the *Timber Construction Manual* (AITC 2013).

Concrete Pier Construction

Concrete deck construction typically consists of a deck slab that provides a working surface and usually bears on and is attached to pile caps that transmit loads to the pile foundation and beams that further strengthen the deck and/or support local loads, such as crane rails and fender systems. The deck slab also performs the important role of distributing horizontal loads via diaphragm action to provide a monolithic structure. Concrete pile caps are usually cast in place. Although precast caps have been used successfully, the means of ensuring an adequate pile head connection often is problematical. Precast concrete pile cap shells subsequently filled with cast-in-place concrete have also been used successfully to reduce formwork on large-scale projects (Padron and Paparis 2004). Pile bent spacing in concrete decked structures usually is within a range of 10 to 20 ft, and slab thicknesses typically range from 8 to 18 in. or more. Pile caps usually are run transversely to the long dimension of a pier, but they may alternatively be run longitudinally. Longitudinal pile caps may be an attractive alternative where there are several railroad and/or crane rails running the length of the pier. Section 7.6 deals with crane rail girder design. The design requirements for railroad trackage are given by the American Railway Engineering Association (AREMA 2014) and by DOD (2004) for U.S. military facilities.

Fig. 7-13 illustrates common types of concrete deck framing; the simplest is a one-way, continuous, cast-in-place slab spanning between pile caps. Where longitudinal girders are used, as for rail girders, two-way slab action may be attained. Not shown in Fig. 7-14 is cast-in-place flat-plate slab construction without pile caps or beams, which simplifies formwork and minimizes surface area and edges exposed to erosive elements. Another common and simple system involves the use of precast/prestressed planks designed as simple spans over which a cast-in-place topping is poured (Gustafson and Long 1981). The topping may be thick enough to contribute to structural action, or it may serve only as a wearing surface. Structural concrete topping must have adequate shear ties between topping and precast plank to ensure full composite action. Differential shrinkage between topping concrete and precast beam may cause tensile stresses in the slab topping and bottom of the beams, which may need to be accounted for when considering other deformation loads. Precast planks are typically placed as simple spans, but the composite system may be made to act as continuous over the supports with sufficient positive reinforcement in the topping slab. Prestress strands from the precast planks should also be extended over the pile cap. A continuous nick joint with filler material should be made along the cap centerline to mitigate cracking. Another deck construction alternative is the use of precast/prestressed planks, generally much thinner than the topping and used as

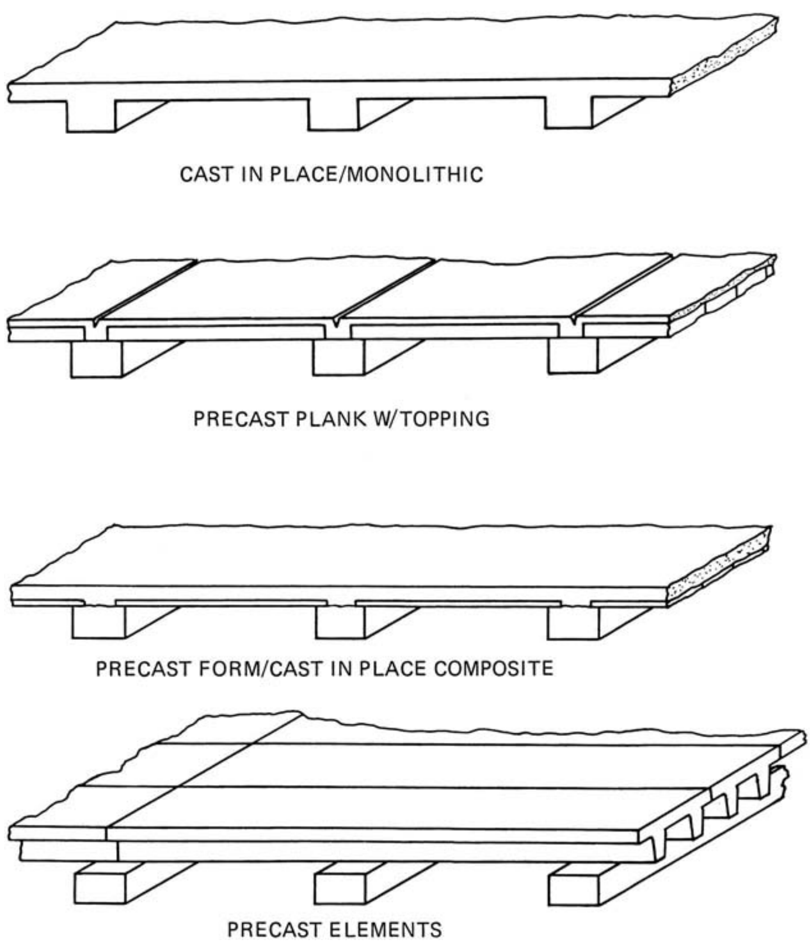


Fig. 7-13. Concrete deck systems

leave-in-place formwork, designed to provide composite action serving as positive moment reinforcement in the completed deck (PCI 2010). Another method of deck construction, more common in trestles and offshore terminals with long spans, is the use of large precast elements, such as double tees or other deep-beam sections, which may be used without any topping.

Where precast planks or elements are used, it is important that adequate lateral ties be provided to prevent differential deflections between members acted upon by concentrated loads and to help ensure overall diaphragm action so that all of the planks act as a unit. This provision is sometimes done by placing posttensioned rods or prestressed strands between adjacent members and across the width of the deck. Adequate corrosion protection of posttensioning rods is essential. Grouted shear keys may also be used to tie adjacent planks together. Where precast composite construction is used, this is usually accomplished by the thickness and lateral

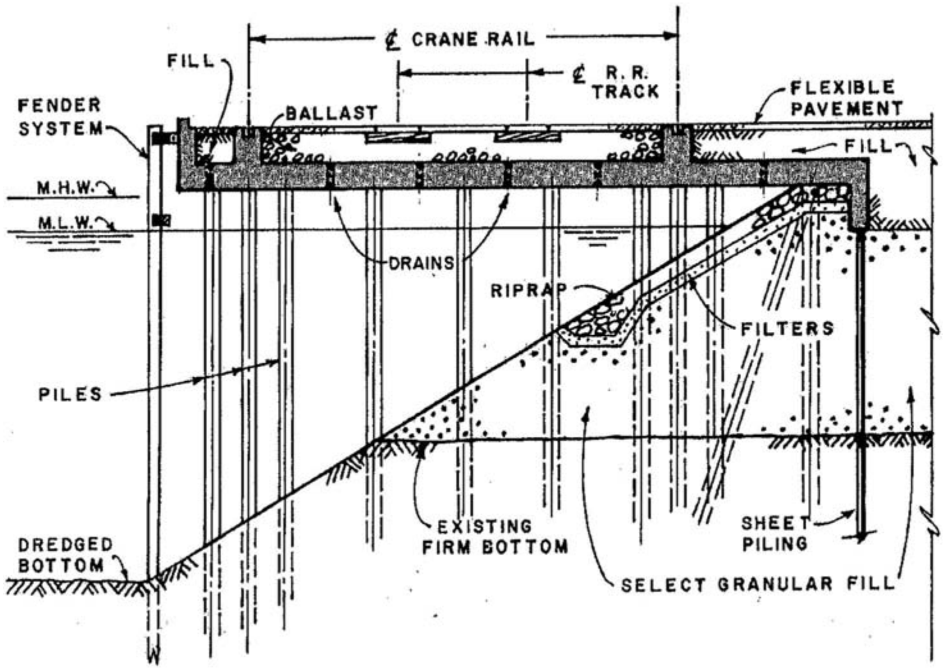


Fig. 7-14. Ballasted deck-type wharf construction

Source: NAVFAC (1980)

distribution of reinforcing steel within the cast-in-place concrete. ACI (1995) provides additional descriptions of typical bridge-deck framing systems and design methods that may also be used in pier construction.

Edge beams should be provided along the exposed pier faces to help distribute berthing and mooring-point loads, provide a flat surface for mounting fenders, and contribute to the overall deck system rigidity. An exception may be lightly loaded access piers without provisions for bringing vessels alongside. Slab edges are especially sensitive to concentrated loads, and thus wheel loads and/or other concentrated loads must be kept within a safe distance from slab edges. Edge beams should generally be of a depth equal to or greater than that of the pile caps. At locations exposed to wave action, care must be taken to avoid creating underdeck void spaces that can trap compressed air, resulting in wave uplift forces on the deck (see Section 4.5). In some instances, vents must be provided, such as via deck drains and/or vent holes through the edge beams.

Deck slabs almost always must be designed to support moving-wheel and other concentrated loads, so they should be reinforced top and bottom in both directions in order to distribute the loads and accommodate the changing sign of the moments with load locations. It also is preferable to use a greater number of smaller, well-distributed reinforcing bars than a smaller number of larger bars of equal area, in order to reduce surface cracking and increase impact resistance. Slabs usually must be checked for punching shear resistance under highly concentrated loads, such as crane outrigger floats. It is often not economical to design slabs for crane float loads; therefore, floats must be spotted over caps or girders, and/or timber mats must be used to help distribute the load, which results in obvious operational limitations. Warren and Malvar (1991, 1993) conducted large-scale laboratory tests and finite-element analyses of one-way continuous concrete deck slabs under concentrated, crane-float loads. They found that for typical Navy piers with spans from 14 to 24 ft, the effective width (E) for patch load distribution was more than 10 ft, considerably larger than given by AASHTO (2002, 2014) provisions. The value of E increased with increasing span and decreased with the load near a support or free edge. Inaba (2007) describes the application of finite element analysis (FEA) to determine safe mobile crane outrigger float loads and travel limits. Appleton (1980) and DHA (1980) provide some guidance in analyzing and designing slabs for heavy concentrated loads. Yield line theory is a useful analytic tool for this purpose as well; refer to Kennedy and Goodchild (2003) for a practical introduction.

For slabs on grade such as may be used at closed fill structures, DOD (2005b), PCA (1996), AASHTO (1993), and ACI (1992) provide useful design guidance. Yield line theory has been applied to the analysis of concentrated loads on slabs on grade as reported by Rao and Singh (1986) and Baumann and Weisgerber (1983). Shentu et al. (1997) applied FEA to the load capacity of slabs on grade. Slabs on grade must be in general thickened along their edges and ends because the slab capacity is greatly reduced here. Corners are especially susceptible. Joints between pours should be dowelled, and allowance for potential settlement should be provided.

Design of rigid and flexible pavements, such as those used in backland staging and storage areas, is outside the scope of this text. Information on pavements, such as block pavers and roller-compacted concrete specific to waterfront construction, can be found in the proceedings of the various ASCE Ports conferences (see Appendix 2).

Additional reinforcing steel typically placed at 45° to main reinforcing at the top and bottom of deck slabs should be provided in addition to the main reinforcement around all unframed deck openings. Additional steel usually is also required around mooring hardware and at fender attachments. Deck surfaces must be sloped to drain, typically on the order of 1/16 in. per foot, and scuppers must be provided along the bottom edge of curbs. Refer to Section 7.8 for further discussion of deck drainage and other features. A minimum 2 in. of clear concrete cover is called for in slabs and beams in the atmospheric zone not subject to salt spray and in the permanently submerged zone, and 2 1/2 in. of cover in tidal/splash zone subject to salt spray (ACI 2014b).

Crack control is an important consideration in the design of marine structures. ACI (2001) recommends a maximum surface crack width under service loads in seawater of 0.006 in. Thermal loads caused by ambient temperature changes must also be considered. Consideration should be given to restraint against thermal expansion and contraction as by batter piles and rigid support conditions.

Deformation loads caused by shrinkage can be considerable, especially on longer structures; creep effects, also known as “rib shortening,” must be considered in the design of prestressed and posttensioned concrete construction. PCI (2010) provides design guidance for shrinkage and creep. All exposed concrete edges should be chamfered from 3/4 to 1 1/2 in. to prevent sharp edges from being chipped off and initiating further spalling. As discussed in Section 3.5, it is important to specify a dense and durable concrete mix.

Another variation of concrete pier construction is the ballasted deck system, as illustrated in Fig. 7-14. Typically 2 to 3 ft of granular fill or crushed stone ballast material is placed over a concrete subdeck and topped with bituminous or other suitable pavement. Such a system is advantageous where heavy concentrated loads may be placed anywhere on the deck surface and/or extensive utility systems are required and can be conveniently routed and buried within the fill. An additional potential benefit is that the massive deck system provides weight to resist batter pile uplift, hence reducing the need for tension piles or other lateral load-resisting system. The added deck mass contributes to the seismic mass and is therefore a disadvantage in high seismic zones. Relieving platform construction, as schematically illustrated in Fig. 7-1, is somewhat similar to a short and heavily ballasted deck system with sufficient length to relieve backland soil pressures on a cutoff bulkhead that may be placed either inboard or at the wharf or quay face. Quay wall-type construction is more common in Europe and along rivers or narrow channels. De Gijt and Broeken (2014) and BSI (2010) provide design guidance for quay wall design and construction.

Fig. 7-15 shows a U.S. Navy double-decked pier that is constructed of precast/cast-in-place composite construction supported on precast concrete piles. This

Fig. 7-15. Double-decked U.S. Navy pier of precast/prestressed composite concrete construction

Source: Courtesy of Appledore Marine Engineering, LLC, file photo arrangement allows utilities to be isolated on the lower deck from other activities on the upper deck, where being at a higher elevation allows more favorable mooring line and gangway angles, as well as unhampered crane operations. Note the extended vertical panels at the lower deck level to accommodate the floating fender unit's movements. A more in-depth description of U.S. Navy double-decked piers is provided by Cole and Farmer (2007). Dubbs et al. (2013) describe the design development of a replacement U.S. Navy fuel oil pier with a new double-decked pier for the largest U.S. Navy tankers.

Composite and Steel Deck Construction

Composite structures usually consist of concrete slabs supported on steel or timber caps or stringers. These methods are not generally desirable in waterfront construction because steel is placed in its worst environment, the splash zone, where it is least accessible, and timber may be subject to decay or compression under heavy loads, which pose severe replacement or repair problems. However, in some applications, such construction may be expedient. Note, however, that in some instances where different materials are used, composite action may not be reliable (i.e., where the caps or stringers and the deck system are structurally independent).

All-steel deck and superstructure framing may be found on offshore dolphin structures in particular and may be of welded or bolted connections for field erection. Deck surfaces are often of heavy-duty deck grating but may be of stiffened plate construction, in which case attention must be given to providing nonslip surfaces as by using pattern plate or other surface treatment.

Dynamic Response

Structure deflections may be enhanced by dynamic effects, especially when the frequency of the applied loading is near the natural frequency of the structure. In particular, marine structures often are subjected to relatively large lateral loads of a cyclic or an impulsive nature, which give rise to large lateral movements compared to those of land-based structures. The dynamic analysis of marine structures follows the same principles as that of any other structure, but special consideration must be given to the nature of the loadings, buoyancy, and the virtual mass of entrained water, damping, and the effect of marine growth. In general, more rigid structures, such as batter-pile-supported piers in shallower water, have higher natural frequencies and hence shorter natural periods (T_n is usually on the order of 0.25 to 0.75 s), thus making them less likely to experience large deflections but more likely to develop high reaction forces from impact. By contrast, more flexible platform structures in deep water, often with $T_n > 1.0$ s, may absorb impacts well but are more susceptible to cyclic wave loading. Note that the structure or its elements also may respond at some harmonic or subharmonic of the natural frequency.

Loadings on marine structures can be broadly classified as follows, in terms of producing dynamically enhanced movements (BSI 2013):

- *Static and long-term cyclic loads*, such as dead loads, stationary live loads, earth pressures, and time-averaged wind and current loads. Such loads are treated by conventional static methods.
- *Impulsive loads*, such as berthing impact; sudden release of mooring lines; wave slam; crane snatch loads; and impact, traction, and braking loads from moving equipment. Such loads normally are accounted for by applying impact factors to the static live load. For more flexible structures, such as offshore berthing dolphins, a dynamic analysis whereby the structure is idealized as a simple single degree of freedom system (SDOF) may be required.
- *Cyclic loads*, such as from regular waves, vortex shedding phenomena, rotating machinery, or traffic vibrations. Dynamic analysis again is carried out by modeling the structure as an SDOF system.
- *Random loads*, such as normal (irregular) wave loadings acting directly on the structure or via the moored vessel, turbulent wind gust effects, and seismic loadings. For nearshore structures, random loads may be approximated by assuming a combination of cyclic and impulsive loads. In offshore structures with water depths approaching and exceeding approximately 100 ft, a more complex spectral analysis may be required. The spectral analysis usually is performed in the frequency domain and requires the development of transfer functions (response amplitude operators, RAOs), as introduced in Section 6.6.

Seismic loads are addressed in the following section.

For an idealized simple SDOF system, the natural period of the structure, T_n, is given by

$$T_n = 2\pi\sqrt{\frac{M'}{K_s}} \tag{7-7}$$

where

M' = effective or equivalent mass of the structure, and

K_s = effective spring constant or ratio of applied load to deflection (F/δ) of the structure.

The quantity M' often is calculated by lumping the distributed masses of the structural elements into a single mass with equivalent static inertia properties. Allowance must be made for marine growth and entrained water on the underwater portions of the structure.

The maximum displacement is given by

$$\delta_{\max} = F/K_s\sqrt{\frac{1}{(1 - T_n/T)^2 + (2qT_n/T)^2}} \tag{7-8}$$

where
T = period of the applied force F, and
q = proportion of critical damping, usually in the range of 0.01 to 0.05 for marine structures of interest herein.

The square root term in Eq. (7-6) is known as the dynamic amplification factor (DAF); if it exceeds 1.2, more exact analytical methods must be used. Added mass and damping factors for marine structures and an overview of dynamic analysis principles can be found in Naess and Moan (2013), Wilson (2002), and CIRIA (1978).

In calculating the value of K_s, it is important that consideration be given to piles with high slenderness ratios $(K_c l_u / r)$ and high axial loads, which may be subject to horizontal displacements at their top connections. For example, consider the case of a cantilevered pile bent where the pile is fixed in the soil at some depth below the surface and encased in concrete at the pier deck such that it is fixed against rotation but allowed to translate under a horizontal force at the deck level. For this case, the theoretical column length factor k_c equals 1.0, and the pile lateral stiffness is given by

$$K_s = F/\delta = \frac{12EI}{l_u^3} \tag{7-9}$$

If the pile is simultaneously under a heavy axial force (P) such as a crane wheel load that is significant in proportion to its critical buckling load (P_{cr}), then the initial deflection, δ, is magnified, and the spring constant (K_s') is effectively made softer, in accordance with

$$K_s' = F/\delta(1 - P/P_{cr}) \tag{7-10}$$

The critical buckling load is given by the Euler equation, as introduced in the discussion of pile slenderness effects earlier in this section, see Eq. (7-3) repeated here:

$$P_{cr} = \frac{\pi^2 EI}{l_u^2} \tag{7-3}$$

A more rigorous analysis involves calculation of the various possible natural frequencies and mode shapes of the pier. For a simple planar bent, the overall sway motion of the bent often is coupled with the bowstring action of the individual piles (Fig. 7-16). A more rigorous dynamic analysis involves solving the general equations of motion using a lumped-mass mathematical model with viscous damping, linearized soil spring, and hydrodynamic forcing function. This model can be expressed in matrix form as

$$(\mathrm{M})\{\ddot{x}\} + (\mathrm{C})\{\dot{x}\} + (\mathrm{K})\{x\} = \{\mathrm{F}\} \tag{7-11}$$

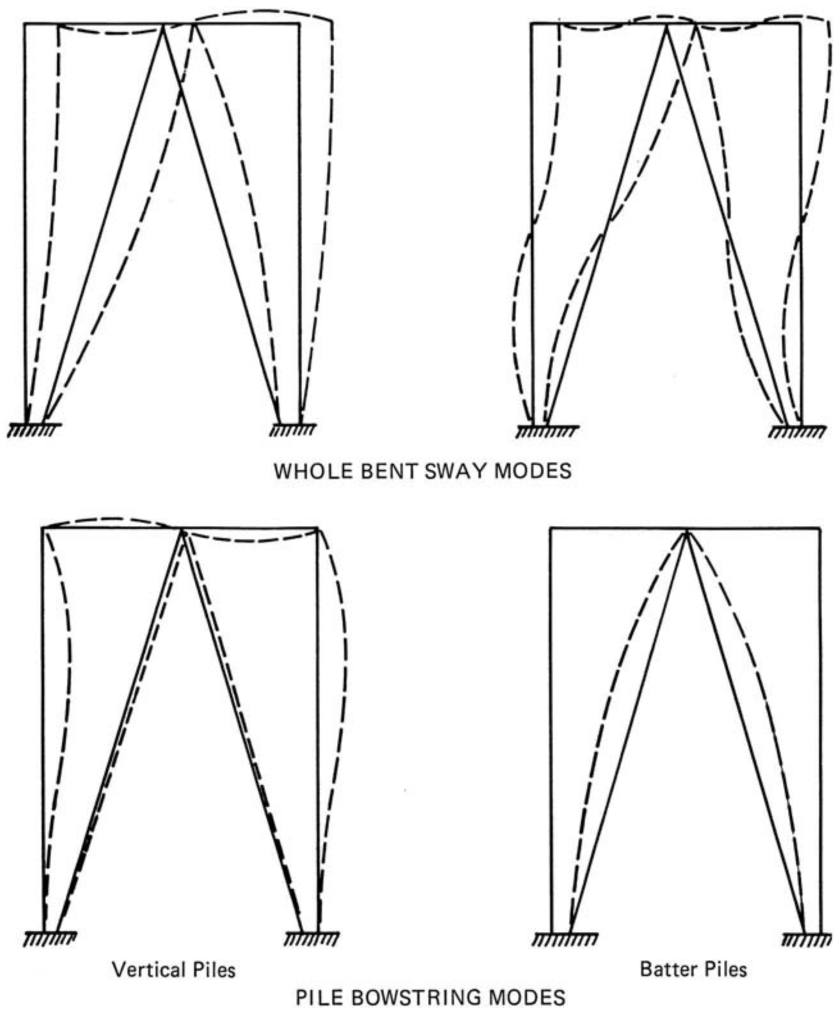


Fig. 7-16. Pier bent mode shapes

where

(M) = total mass matrix of the system;
(C) = damping matrix;
(K) = structural stiffness matrix;
{x}, $\{\dot{x}\}$, and $\{\ddot{x}\}$ = nodal displacements, velocities, and accelerations; and
{F} = forcing function.

The equation is solved either in the time domain by direct integration over a given time interval or more often in the frequency domain using response-spectrum techniques. Computers are mandatory for this type of analysis.

Fatigue Design Considerations

Structures subjected to continuous cyclic loading may suffer a reduction in strength known as fatigue, which manifests itself in a process of crack initiation and subsequent crack growth. As a general rule, nearshore/shallow-water structures and, in particular, concrete waterfront structures do not need to be designed to fatigue criteria. Tubular steel structures in deep water exposed to wave loading are particularly susceptible, although it is important to check any structure with significant dynamic amplification of loads for fatigue effects. In practical design, it is customary to compare a structure's intended design life with its predicted fatigue life as limited by "hot spot" stresses (i.e., areas of high local stress concentrations) versus number of load cycles. In general, the design fatigue life of each joint and member should be twice the intended service life of the structure. One practical problem of fatigue design is uncertainty in determining the structure's lifecycle load history.

Fatigue design criteria are frequently presented in the form of an *S*–*N* diagram, which plots maximum hot spot cyclic stresses or stress range versus number of load cycles to failure based upon empirical data. In the saltwater environment, corrosion can enhance the fatigue failure mechanism. Fig. 7-17 shows the results of corrosion fatigue testing on the welded joints of tubular offshore structures plotted on an *S*–*N* diagram. The API-*X* curve represents the recommended design criteria for as-welded tubular connections subject to random loading in seawater. The scatter plots show the corrosion fatigue life of tubular joints made with various welding procedures; some results fall below the *X* curve. Refer to Ramachandra Murthy et al. (1994) for further description of these results and discussion of corrosion fatigue of tubular joints in seawater. Fatigue analysis of structures subject to wave action is complex, and the reader should refer to API (2014) for design guidance and to ABS (2003) for fatigue analysis of offshore structures in general. An alternative to the use of *S*–*N* curves is the application of linear elastic fracture mechanics (LEFM). Li et al. (2011) describe the use of the LEFM method in the fatigue reliability assessment of the circumferential butt splice welds of berthing monopiles at an iron ore terminal subject to frequent berthing impacts.

An in-depth case study of steel-pipe-pile fatigue failures during the construction of an offshore tanker terminal in the Arabian Gulf was provided in a series of papers by Weidler et al. (1987). Freestanding vertical piles failed at circumferential butt splice welds near the mudline because of fatigue damage, the principal mechanism of which was a dynamic response to vortex shedding under storm wave conditions. These studies show the importance of considering that fatigue damage might occur during the construction phase. In this case, all freestanding piles that had been exposed like the failed piles had to be cut off at the mudline and were overdriven by larger diameter piles as part of the repair program.

In general, fatigue effects can be minimized by sound connection details that avoid stress concentrations. Fillet welds should be avoided and full-penetration, groove-type welds should be used where the structure is subject to repeated load

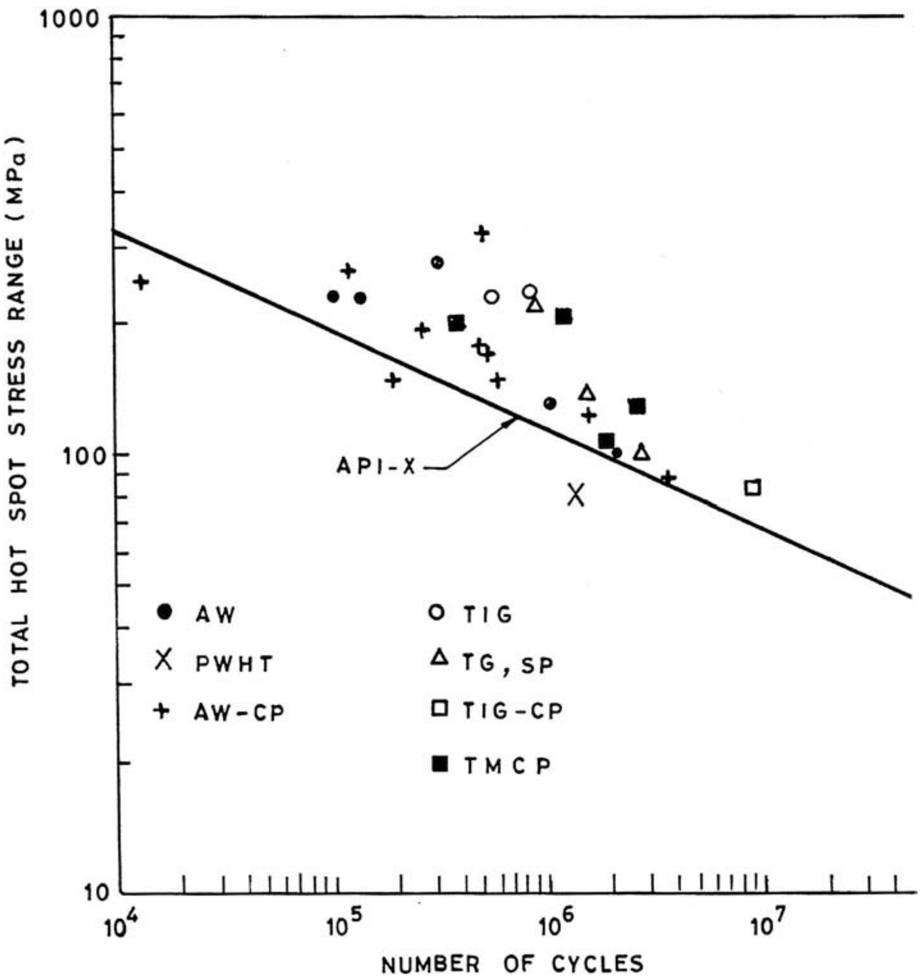


Fig. 7-17. *S–N* diagram showing corrosion fatigue test results for tubular joints in offshore structures (100 Mpa = 14.5 ksi)

Source: Ramachandra Murthy, et al. (1994), copyright ASCE

cycles. Within the splash zone in particular, corrosion fatigue can result in an especially rapid propagation of fatigue cracks, so the use of corrosion-resistant materials and details helps reduce cumulative fatigue effects.

Fatigue is not normally a factor in the design of reinforced concrete port structures but could be important under certain circumstances, where structures are exposed to continual repeated heavy loads, such as a severe wave climate, high-use barge, or ferry berthing structures. Marshall (1990) provides a rigorous treatment of fatigue in concrete marine structures, and ACI 357R (1984) offers general design guidance for offshore concrete structures.

Solid Fill Piers and Wharves

Solid fill berth structures can be generally classified into two main types: *gravity walls*, which depend upon their own mass and bottom friction to resist overturning and sliding, and *anchored walls*, which require tieback anchors and/or anchor piles to gain stability and resistance under horizontal loading. Gravity walls include stone masonry and massive concrete walls; sheet pile cell walls and precast concrete; and block, box caissons, and cantilever L-walls. Anchored walls include sheet pile bulkheads and relieving platform structures that are anchored by batter piles, tiebacks, or friction slabs, and/or some combination thereof. Fig. 7-1 illustrates various solid fill structure types. Sheet pile cells and bulkheads, as well as geotechnical aspects of solid fill structures in general, are addressed in Sections 8.5 and 8.6. General failure modes are illustrated in Fig. 8-8. Principal modes of failure for gravity walls include sliding, overturning, foundation failure, and deep slip/slope stability failure. The following paragraphs focus on some structural and functional design aspects of precast concrete elements typically used in gravity-type wall construction. This type of construction is far more common outside of the United States.

Caissons and Precast Elements

Large-scale precast concrete units, including box and pneumatic caissons, plain and buttressed L-walls, and cellular and solid blocks, are often used in massive waterfront quay wall construction. All of the above types of precast elements require a carefully prepared and leveled foundation base. Joints between abutting units require careful design, both to allow for some differential settlement and to form a tight soil-retaining seal. Application of L-wall and caisson elements, in particular, usually is economical only on large-scale projects where many units are required and where sufficient land staging areas and/or dry dock building basins are located nearby. L-wall elements may be constructed as simple cantilever L-shaped sections for walls up to around 25 ft high, or they may be buttressed with *counterforts* to stiffen the base and stem, in which case wall heights of around 60 ft are practical. Individual units typically range from about 10 to 40 ft wide, depending upon the capacities of handling and placing equipment.

Box caissons can be built to heights of 60 ft or more; plan dimensions of around 100 ft are common, and maximum length dimensions as limited by longitudinal stresses are on the order of 300 ft or more. Box caissons usually are built in dry docks or building basins formed by temporary earth berms or sheet pile cell walls and are floated into position where they are ballasted down and filled with earth or stone fill. Aspects of the design of concrete “float-in” structures are covered in detail in ACI (2010). As illustrated in Fig. 7-18, box caissons usually are compartmented internally for strength and ballast control. Also, internal compartments can be filled differentially when in position to help reduce peak overturning toe pressures caused by lateral earth pressures. Caissons also may be of the pneumatic type with recessed

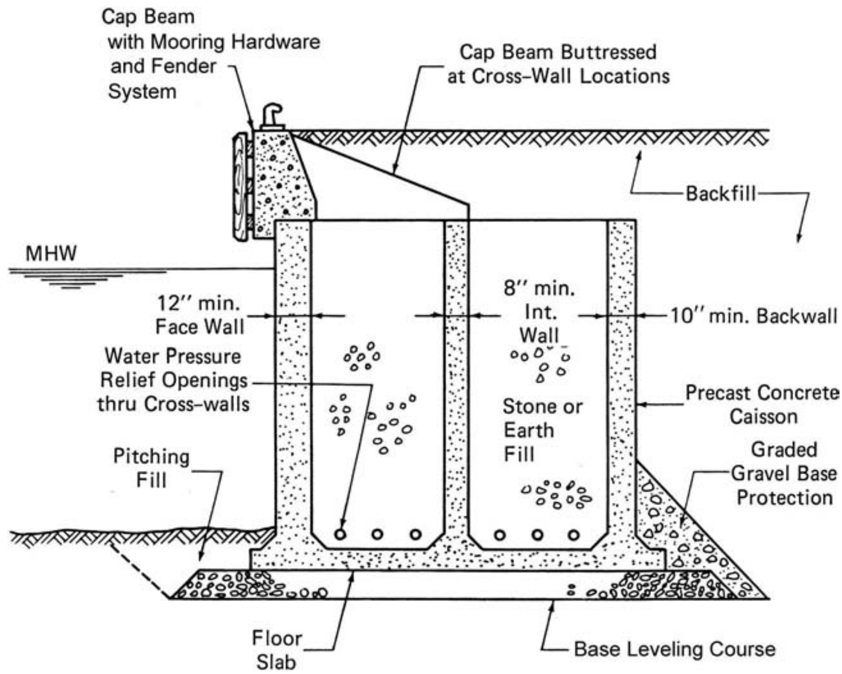


Fig. 7-18. Elements of precast concrete box caisson design

bottoms, or they may be of open construction without a bottom slab. These types are typically slid into position from the land side. Open-bottom “monoliths” have open cell compartments that are typically infilled with concrete after positioning.

Stacked precast blocks with shear keys between courses are a durable form of construction that generally requires divers during construction and must be founded on firm soils to avoid settlement under their relatively great weight. Individual block weights are limited by the equipment available to handle them and may range from 5 to more than 200 tons. The largest size that can be practically placed is desirable for structural integrity as well as minimizing joints and placement operations. To help minimize weight and hence cost, blocks can be placed in a “broken-back” or humpback pattern in section such that the lower portion of the wall slopes landward, resulting in a smaller active soil wedge where pressures are highest, and leans forward at the top, where pressures are lower. The stability of such a configuration is improved over a straight vertical back wall because the center of gravity of the wall is moved landward (Sadrekarimi et al. 2008).

All types of precast concrete element walls usually have a continuous cast-in-place concrete capping beam to help tie adjacent units together and form a smooth, continuous face into which mooring and fendering hardware can be cast. The capping beam should be placed after any expected initial settlement of the precast units has occurred. Capping beams, in general, should be designed as elastic beams on continuous elastic foundations and must be strong enough to distribute local berthing and mooring forces and to accommodate any long-term differential settlements between precast elements. Capping beams also may serve as crane rail trackage. The capping beam typically performs a soil-retaining function as well. UFC 4-151-10 (DOD 2001) calls for expansion joints at 300 to 400 ft maximum for concrete quay walls. Expansion joints are not generally required in capping beams of sheet pile structures.

In the structural design of precast elements, the construction, handling, and/or towing and placement stresses may be more severe than the in-service design load stresses. In-depth treatment of the structural design and applications of precast elements in the construction of quay walls can be found in the handbook *Quay Walls* (De Gijt and Broeken 2014). General design guidance is in BSI (2010), EAU (1996), and OCADI (2009), and several case history design examples can be found in PIANC (1986). PIANC (2005) provides a catalog of prefabricated concrete elements for marine construction. Geotechnical aspects of precast element design include resistance to sliding and overturning, and the evaluation of bottom bearing pressures and lateral earth forces, as discussed in Chapter 8.

7.4 Seismic Design Considerations

Seismic design requirements for pile-supported piers and wharves are covered in the ASCE Standard *Seismic Design of Piers and Wharves*, ASCE/COPRI 61-14 (ASCE 2014). Seismic design guidance for waterfront retaining structures, for which there are currently no specific standards, can be found in the NCEL Technical Report, R-939, by Ebeling and Morrison (1993). Brodbaek and Laursen (2010) provide an overview of the state-of-the-art seismic design of caisson waterfront structures. The State of California building code includes specific seismic requirements for marine oil terminals (MOTS) (MOTEMS 2011), and the Ports of Los Angeles and Long Beach have their own specific seismic design requirements for container wharves (POLA 2010, POLB 2012), respectively. PIANC (2001), *Seismic Design Guidelines for Port Structures*, provides overall design guidance for port structures. The reader should refer to Section 4.6 of this book for background on code requirements and an introduction to the nature of seismic loads. Rigorous treatment of seismic design of marine structures is beyond the scope of this text because of the breadth of this subject. The following discussion is intended to highlight some important seismic design considerations and direct the reader to more detailed treatment. Case history design examples can be found in the various proceedings of the ASCE/COPRI Ports conferences and PIANC congresses (see Appendix 2).

As introduced in Section 4.6, the seismic design of piers and wharves is in accordance with specified performance levels for the operating-level earthquake (OLE), contingency-level earthquake (CLE), and design-level earthquake (DE) associated with minimal damage, controlled damage, and life safety, as further defined in the referenced standards. The specified performance level is also generally associated with an importance classification of high, moderate, or low, based upon the economic and life safety consequences of damage. In addition to the direct effects of ground accelerations, the possibility of soil liquefaction and associated lateral spreading, cyclic degradation of weak soils, slope stability, kinematic loading of piles caused by soil movements, and the effects of fault rupture must also be evaluated. Two basic approaches to seismic analysis and design can be defined as force-based design (FBD), wherein structures are designed to withstand prescribed design loads while remaining within prescribed stress limits, and displacement-based design (DBD), wherein an anticipated displacement of the structure is determined for the design-level earthquake motions and the structure is designed to perform to specified criteria under the given displacement. FBD is the simplest to apply and is generally acceptable for low seismic classification and lower ground accelerations, whereas DBD is acceptable and in general is preferred for all cases. FBD may consist of a simplified equivalent lateral load analysis by applying a seismic response coefficient (C_s) to the seismic weight of the structure to determine a base shear. The determination of C_s in turn depends on application of response modification (R) and deflection amplification (C_d) factors as a function of the type of construction, as given in ASCE (2010, 2014) in general for building structures. A modal response spectrum analysis may be conducted as a refinement of this procedure. DBD takes into account the inelastic properties of the construction materials and typically involves comparing a displacement capacity determined by a nonlinear pushover analysis to a displacement demand associated with one of the previously described seismic hazard levels. Alternatively, a more elaborate nonlinear time history may be performed. This method requires obtaining multiple suitable ground acceleration records.

Load combinations for seismic design do not need to include berthing and mooring loads but do include an increase and decrease in dead load to account for vertical accelerations, usually one-half of the PGA, plus 10% of the uniform live load, plus earth pressure loads, plus the horizontal earthquake loads taken as 100% in one direction with 30% in the orthogonal direction. Torsional effects must also be considered, and for many applications a dynamic magnification factor (DMF) may be determined to account for both torsional and orthogonal effects in lieu of the more rigorous load cases. In some instances, a greater percentage of live load, ranging from around 20% to 50% or more (Smith-Pardo and Ospina 2013), may be more appropriate than the 10% ASCE (2014) requirement. The seismic mass to be considered includes the entire structural self-weight, plus any permanently attached equipment, plus 10% (or more) of the uniform live load. The top one-third of the length of piles measured from the bottom of the deck to five pile diameters below grade is often lumped with the pile deck mass for analysis. The hydrodynamic "added mass" of the piles must be considered for piles exceeding 2 ft in diameter. The effects of rail-mounted cranes are addressed in a following paragraph.

An important goal in seismic design is to achieve adequate ductility to avoid sudden catastrophic failure. Decks are required to be *capacity protected*, such that ductile failure in the pile tops or connection through plastic hinging is the desired failure mechanism. Therefore, strain limits associated with given design levels are specified in the referenced standards. The flexural and torsional rigidities of concrete sections must be properly represented by an effective moment of inertia that reflects the degree of cracking before the yield limit state is reached. Material properties used in analysis are modified "expected" strengths, as specified in the standards. The use of batter piles in high seismic zones is generally discouraged because of the extremely high pile loads and resulting deck uplift forces. Seismic isolation systems may be provided in some instances (ASCE 2014), and Leal et al. (2013) provide a case history analysis of seismic isolation systems on pile-supported wharves.

In displacement-based design, analysis and design of seismic loads are performed for each seismic hazard and performance level, and the displacement capacity must be greater than the displacement demand for each case. Displacement capacity is normally determined by performing a nonlinear static pushover analysis and the demand by performing a modal response-spectrum analysis. The inelastic material properties must be accounted for as specified in the referenced standards, and the plastic moment capacity of the piles (M_p) is determined by moment-curvature analysis (M-ϕ), from which the plastic rotation (θ_p) is also determined. The plastic rotation and plastic hinge length defined in the referenced standards are then used to convert the pile moment curvature to a force-displacement or moment-rotation relationship for the pushover analysis. Fig. 7-19 schematically illustrates the elements of a pushover analysis on a wharf using the depth-to-fixity method to model the piles within the soil. A more accurate model would use p–y springs instead. As the more highly stressed piles develop plastic hinges in succession, the wharf becomes less stiff and a new effective spring constant for the nth iteration ($K_{e,n}$) can be obtained from the secant stiffness slope of the force-displacement curve. This procedure is described as the substitute structure method in ASCE (2014). Capacity and demand analysis in general are discussed in greater detail by Ferritto et al. (1999), Ferrito (1997), and Werner (1998), and a case history design example for the displacement-based design of a U.S. Navy carrier pier has been presented by Klusmeyer and Harn (2004) and of a seismic capacity evaluation of a container wharf by Vahdani et al. (2007). Nonlinear time history analysis accounts for both demand and capacity analysis simultaneously (Blandon 2007).

The pile-to-deck connection is critical to meeting seismic design requirements, and selection of connection type should be based upon structure type, seismic demand, pile type and length(s), soil stiffness, postearthquake reparability, and consideration of other load sources, such as berthing and mooring, and of corrosion and lifecycle costs. Pile-to-deck connections may be of the full-moment or partial-moment capacity type. Fig. 7-20 illustrates some generic pile head connection types, including relative lengths of plastic hinge zones for steel pipe and prestressed concrete piles. Specific design requirements for these types of connection are given in ASCE (2014), as well as plastic hinge lengths and related dowel strain penetrations. Plastic hinge lengths within the soil can be taken as $2D$ for both steel pipe and prestressed concrete piles.

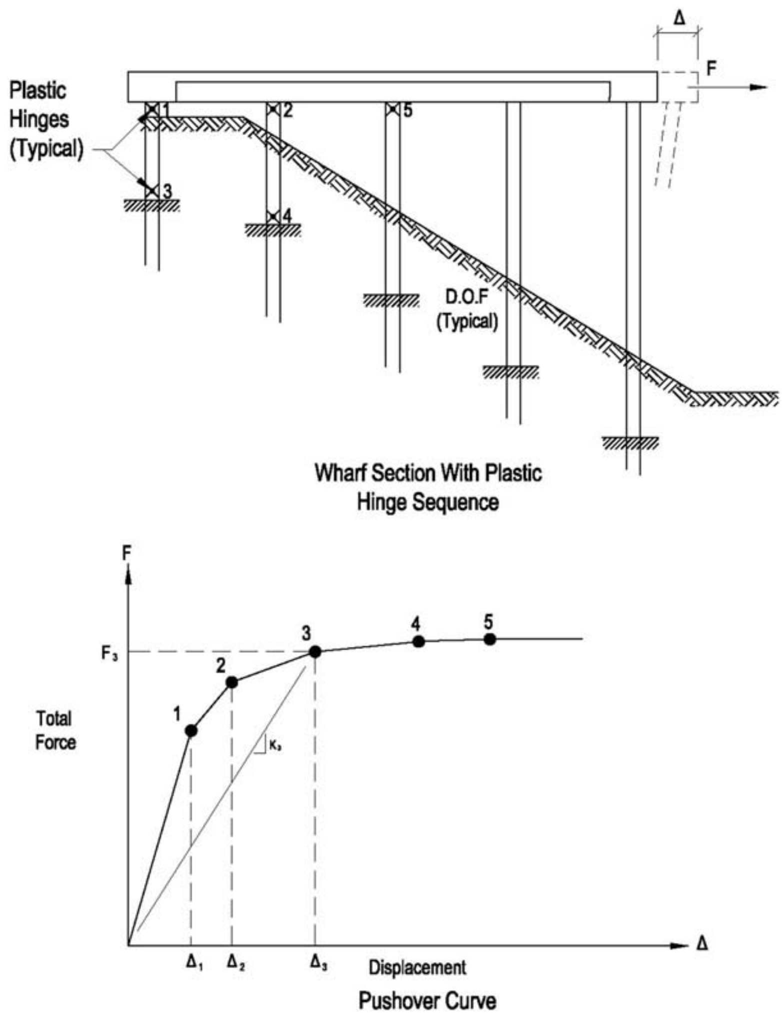


Fig. 7-19. Pushover analysis schematic wharf section and force displacement curve

Source: Adapted from POLA (2012)

Ancillary, or nonstructural, components are components of a pier or wharf that do not contribute to the strength or stiffness of the structure and include cranes, loading arms, pipelines, and pipe supports in particular. Container crane mass need not be considered in the wharf seismic analysis if the crane natural period is at least twice the wharf period in the direction considered or if the crane mass at or below the crane portal beams is less than 5% of the tributary wharf mass, where the tributary wharf mass is based upon the lesser of the wharf length per crane divided by the number of cranes, or 300 ft, and the mass of the wharf deck and upper one-third of the pile length measured down to $5D$ below grade. Jaradat et al. (2013) present the results of a nonlinear time history analysis of crane–wharf interaction at POLB where

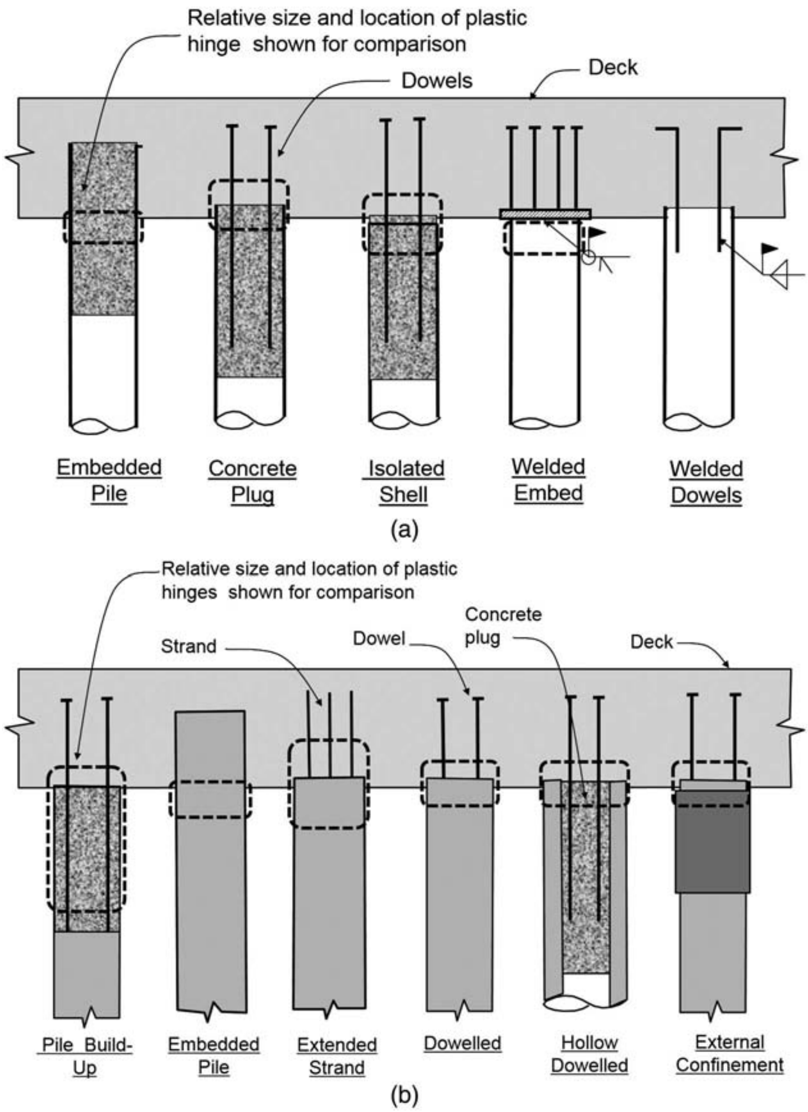


Fig. 7-20. Pile head connection types and plastic hinge zones for seismic analysis

Source: ASCE (2014); copyright ASCE

this criterion was not met. The stability and structural integrity of the crane itself must also be considered. Typical failure modes include derailment, detachment of bogies, buckling of legs and collapse, overturning, and rupture of clamps and anchors of secured cranes. Soderberg and Jordan (2007) review the seismic response of jumbo container cranes and provide recommendations to limit damage and prevent collapse. They note that cranes have typically been designed to lower design requirements than wharves are designed to, with a 0.2g lateral seismic loading and elastic response associated with zero damage. They further note that older and smaller cranes lift off the rails because of tipping at around 0.2 to 0.3g and that the crane support structure is inherently capable of resisting the associated bending moments, as compared with much larger contemporary cranes, which may not lift off below 0.5 to 0.6g, resulting in much higher moments on the support structure and greater likelihood of collapse. Srivastava (2013) addresses the seismic response of nonstructural components on piers and wharves and reports that for components such as loading arms, gangway towers, pedestal cranes, and light poles that have a mass ≤1% of the wharf mass, the component response can be decoupled from the overall structure response, interaction need not be considered, and the component can be treated as a simple SDOF system. Serviceability of quay walls with pipelines and conveyors may be seriously affected by differential settlements. The reader is referred to ASCE (2014) for additional ancillary component requirements.

The below-deck bottom slope or embankment below wharves must be checked for overall slope stability and lateral ground deformations under seismic loading. Initial estimates of the free-field seismic ground deformations may be determined using the Newmark sliding block analysis (Newmark 1965), which can also be used to check pile kinematic loading. The POLA (2010) and POLB (2012) codes require that a minimum backland surcharge of 250 lb/ft^2 be applied within 75 ft of the inner wharf margin, a surcharge of 1,200 lb/ft^2 for static slope stability, and 800 lb/ft^2 for pseudostatic and postearthquake slope stability analysis over the remaining backland area. More elaborate finite element (FEM) or finite difference (FDM) models may be required to carry out 2D or 3D dynamic analysis to properly account for soil–structure interaction (SSI). Examples of such computer programs include the 2D, FDM FLAC (ITASCA 1999) and the 3D, FEM PLAXIS (Brinkgreve et al. 2013), which can be applied to a wide range of SSI problems, including progressive liquefaction and soil–pile interaction, as well as slope stability. An example application of FLAC analysis to the evaluation of an existing wharf in the Port of Oakland, California, in the 1989 Loma Prieta earthquake is provided by Singh et al. (2001). FDM and FEM programs are also often applied in the analysis of caisson gravity structures and sheet pile bulkheads as well.

An interesting design problem arises in the design of dolphins or piers to which floating dry docks or relatively large floating structures are permanently moored. Assuming that the connection is relatively rigid and that the dry dock is massive, extremely large forces may be developed at the connection if the pier structure is subjected to strong ground motions. The mass inertia of the dry dock plus the added mass of entrained water is so large that it is essentially unmovable under short-duration ground shaking. In order to prevent damage to the pier structure and/or connections, a flexible connection or release mechanism (fusible link) must be provided, as described by Keith et al. (1986) and by Mazurkiewicz et al. (1977).

7.5 Dolphins and Support Structures

Dolphins are discrete structures designed primarily to accommodate the lateral forces associated with vessel berthing and mooring. Some dolphins, referred to as *mooring dolphins*, are designed strictly to secure a vessel's mooring lines. *Berthing dolphins* are designed primarily to absorb the impact of berthing vessels and usually are fitted with a fender system and sometimes also with mooring bollards. Dolphins can be further classed as being *flexible*, such as timber pile clusters and cantilever piles, where the dolphin's own deflection can be used to absorb berthing energy, and *rigid*, such as batter pile or braced-frame dolphins and cells and caissons, with limited deflection, where a fender system is required to absorb berthing energy. Rigid dolphins are preferred for mooring applications in order to reduce changes in mooring line lengths under load. Dolphin structural types vary largely with water depth, bottom conditions, and vessel size. Fig. 7-21 illustrates a progression of dolphin types, showing their approximate ranges of water depth and vessel displacement.

Timber pile clusters are a common feature of a working waterfront, usually constructed of groups of 3, 7, 19, or sometimes 30 piles secured with wire rope wrappings. The center pile of the group typically projects approximately 3 ft above the other piles to accommodate mooring lines (Fig. 7-22). Timber pile clusters

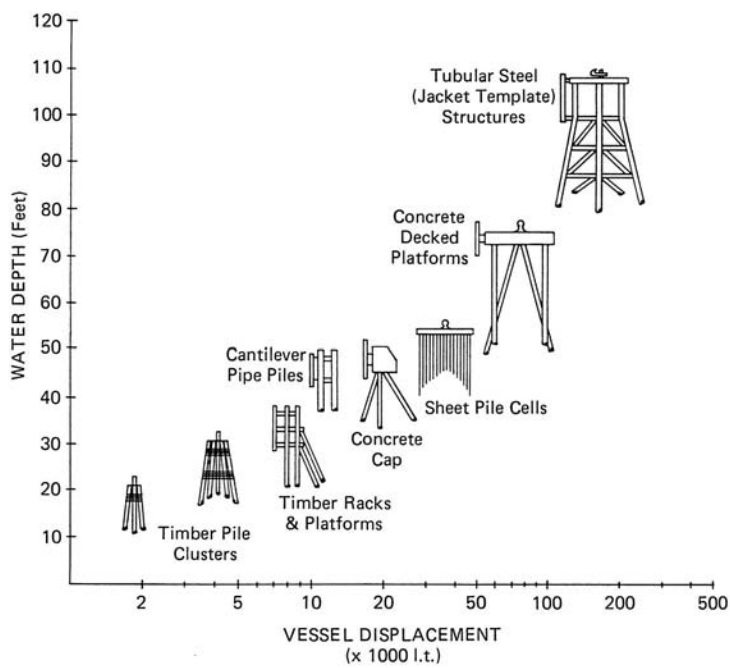


Fig. 7-21. Ranges of application of dolphin types

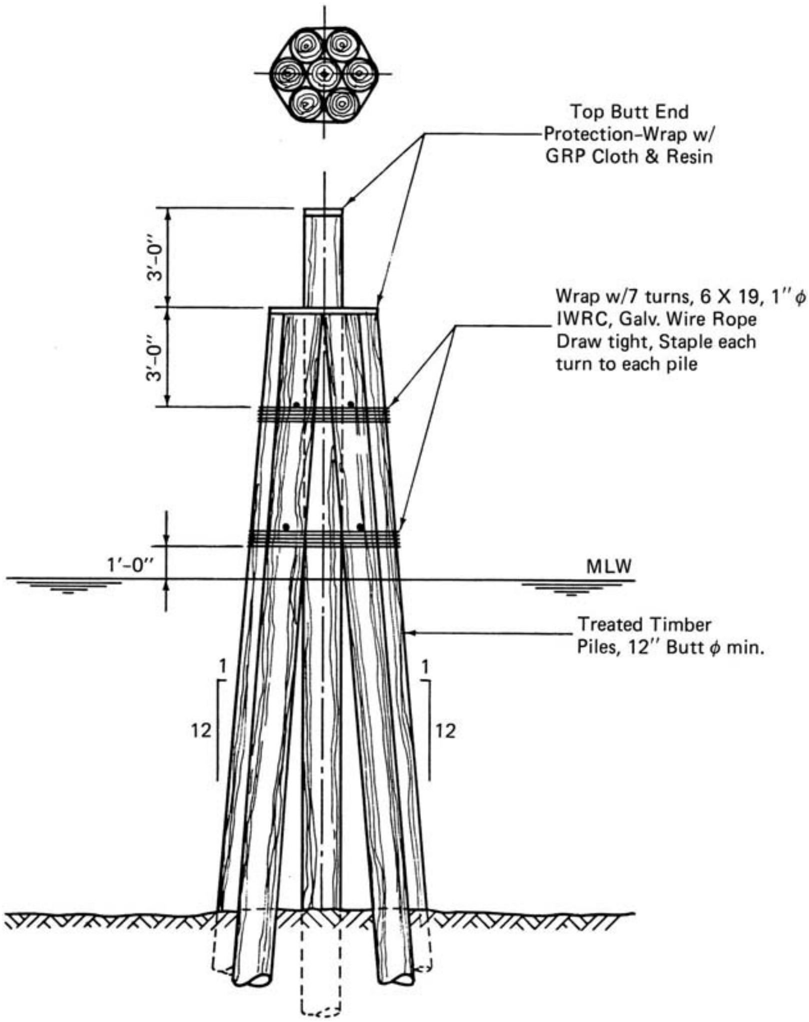


Fig. 7-22. Traditional seven-pile timber cluster dolphin

provide relatively low energy absorption, and their lateral load capacity usually is limited to 10 to 15 tons maximum. They generally are suitable and certainly are economical in water depths of up to about 35 ft and for vessels of up to 2,000 to 5,000 dead weight (DWT). The horizontal load capacity of timber pile clusters is carried mainly by axial loads in the piles, and their ultimate capacity thus is limited by individual pile pullout capacities. Also, the center king pile may take most of the load initially and fail before the surrounding piles develop their capacity, thus reducing the overall capacity of the timber pile cluster. Energy absorption is primarily via bending. Elms and Schmid (1965) show that both load capacity and energy absorption can be increased by providing the proper shear flexibility at the pile heads. They present graphical data and a semiempirical formula for calculating the dolphin energy and maximum horizontal load capacities for 7-, 19-, and 30-pile clusters. With repeated use over time, many timber cluster dolphins suffer from soil shakedown and do not spring back to their original upright positions.

Timber racks and platform structures with both battered and vertical piles and intermediate framing (see Fig. 5-14) have been used for vessels up to around 17,000 DWT and may have a lateral load capacity on the order of 40 to 50 tons. As with pile clusters, however, their ultimate capacity is limited by pile pullout capacity, and their energy absorption is relatively low. Cantilevered pipe piles and, occasionally, prestressed concrete piles driven singly or in groups offer greater energy absorption than these structures and may be the most economical solution where soil conditions permit.

Cantilevered steel pipe piles (Fig. 7-23) may be used in water depths of up to approximately 50 ft and sometimes more. Because they exhibit linear elastic behavior, the berthing energy to be absorbed can be related directly to the pile reaction force (F_R) and the deflection (δ). Equating the energy of impact absorbed by the dolphin (E_F) to the work done by the pile deflection gives $E_F = F_R\delta/2$; and noting that the spring constant $K_s = F_R/\delta$, it can be shown that

$$F_R = \sqrt{2K_sE_F} \tag{7-12}$$

For a single fixed-end cantilever pile,

$$\delta = \frac{F_R l_u^3}{3EI} \tag{7-13}$$

where

E and I = pile modulus of elasticity and moment of inertia, respectively, and
l_u = unsupported length from the point of fixity to the point of load application.

The spring constant thus is given by

$$K_s = \frac{3EI}{l_u^3} \tag{7-14}$$

Note that if the dolphin is fitted with resilient fendering, the value of K_s must be adjusted for springs in series, as described in Section 5.6.

It can be further shown that the pile deflection is related to the bending stress (f_b) by

$$\delta = \frac{2f_b l_u^2}{3DE} \tag{7-15}$$

where D is the pile diameter.

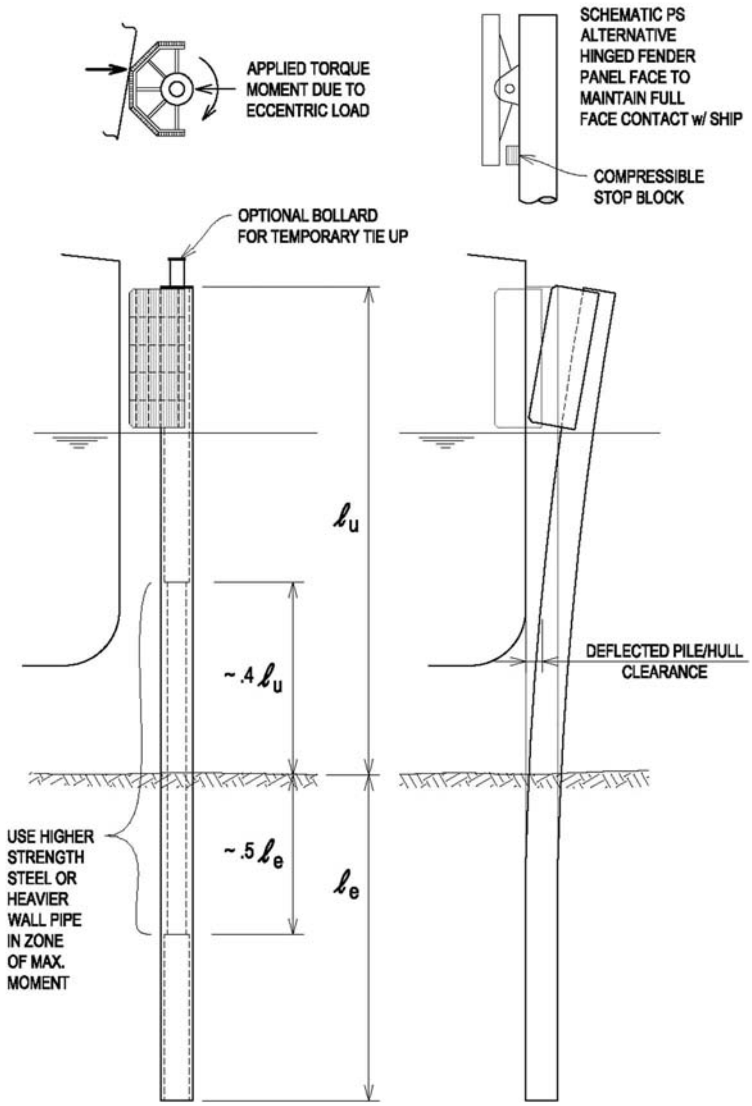


Fig. 7-23. Steel monopile dolphin

The allowable bending stress in a steel pipe is often taken as two-thirds of the yield stress (F_y), implying a factor of safety (FS) of 1.5 for the material's yielding. Some engineers may apply the FS of 1.5 to the ultimate berthing energy, which varies inversely as the square of the deflection, resulting in an allowable stress in the steel of $\sqrt{0.67}F_y = 0.82F_y$ (PIANC 2002). The pipe pile wall must be sufficiently thick that local buckling does not occur. Fatigue may also need to be considered for cantilevered steel pipe dolphins (Li et al. 2011). Strong currents may result in vortex-shedding oscillations (see Section 4.5). At sites with high corrosion rates and frequent use, especially under wave action, allowable stresses may be decreased about 50% to allow for fatigue effects. The energy capacity of a flexible pile is proportional to the square of the steel stress and linear to the wall thickness, thus favoring the use of high-strength steel and substantial wall thickness for high-energy absorption. The use of high-strength steel, at least in the zone of high bending stress, increases the energy absorption for a given size pipe and water depth. The use of stepped pipe wall thickness allows the moment of inertia to vary along the pipe in accordance with the applied bending moment, thus resulting in increased deflection and, hence, greater energy absorption. Splices are typically circumferential full-penetration V-bevel girth butt welds, usually made without backing.

The application of plastic design to the optimization of flexible dolphins has been presented by Bruijn et al. (2005), who noted the trend toward allowable stresses approaching and equaling the yield stress based on the assumption that some permanent deformation caused by plastic yielding does not instantly lead to ultimate failure. EAU (1996) and PIANC (2002) allow some permanent deformation to occur before ultimate failure, resulting in up to about a 30% increase in energy absorption at failure. Bruijn et al. (2005) importantly note that with larger dolphins with increasing pile diameter, the pile diameter to wall thickness ratio (D/t) is usually reduced and sensitivity to local pile wall buckling and ovalization increases. *Ovalization* results from a combination of bending stress and external soil pressure and enhances local buckling such that buckling usually occurs first and is the primary mode of failure with $D/t > 80$. Their analysis also demonstrated that for $D/t > 40$, a check of critical buckling strain should be conducted as part of the design verification. For lower D/t ratios, the pipe fails in the usual flexural bending stress mode. The possibility of local wall buckling before a pile reaches its limit state design by flexural failure can be offset by using increased wall thickness in the high moment areas and/or by partial filling with sand or concrete to prevent ovalization.

Suitable subsurface soil conditions are essential to the application of cantilevered piles, and the foregoing discussion assumes that the pile capacity can be fully developed by the subsurface conditions. Kray (1976) has presented some novel cantilever pipe pile applications. Cantilever dolphins that can be loaded only from one direction can be driven at a slight batter toward the direction of impact, resulting in an effective prestressing that increases the energy absorption potential of the pile. Cantilevered piles can also be driven in groups and/or in rows to form berthing beams. They can also be fitted with torque arms to gain additional energy absorption by applying a torque moment in adjacent piles as the dolphin deflects under an eccentric load application. In the design of flexible dolphins in general, maximum deflections must be checked to ensure that vessels do not contact lower portions of the piles under design load conditions and/or that catwalks or gangways remain supported. Kluwmeyer et al. (2007) report on a systemwide performance study of flexible steel dolphins for the Washington State ferry system. Cantilever monopiles

Fig. 7-24. Concrete cap breasting dolphins with resilient rubber fender units and face panels

are also frequently used as guide piles to moor floating docks and as support piles for fixed aids to navigation, as described in the following discussion.

Where more energy absorption is required and lateral forces are higher, groups of battered and vertical piles may be capped with a massive concrete cap and fitted with resilient fender units. An example of such dolphins is shown in Fig. 7-24. In this case, the center of gravity (c.g.) of the concrete cap is designed to be aligned vertically with the center of action of the batter piles so that its weight reduces the uplift on the tension piles. In water depths approaching and exceeding approximately 50 ft and where mooring hardware is to be installed, concrete-decked platforms may be constructed, such as the dry dock mooring platform shown in Fig. 7-25. Fig. 7-26 summarizes the forces acting on a typical berthing and mooring dolphin. Note that, in addition to the horizontal forces caused by berthing and mooring line loads, vertical and longitudinal rubbing forces and vertical mooring line components also must be considered. The vertical lead of mooring lines should be considered in setting dolphin deck elevations. Loads applied to the dolphin that do not act through its c.g. result in torsional moments that must be resisted by the pile foundation. In the design of multipiled dolphins, attention must be given to pile spacings and possible reduction of pile capacities caused by group effects, as described in Section 7.3.

In water depths of up to 40 to 50 ft where rock or firm bearing strata are near the surface, a circular sheet pile cell with a concrete cap may be the most economical

Fig. 7-25. Mooring-platform-type dolphin with concrete deck and steel pipe pile foundation

solution (see Fig. 8-20).The cell must be designed to resist sliding and overturning, and loss of fill at the heel caused by tilting and interlock tensions must be checked for hoop stresses caused by the confined fill. Fig. 7-27 summarizes the soil and structure (SSI) resisting forces acting on a typical sheet pile cell under lateral loading. Cells make excellent warping or turning dolphins and are capable of resisting large lateral loads. Although cells possess excellent impact resistance, they must be fitted with fendering to obtain suitable energy absorption, and their geometry may make the installation of high-energy resilient fenders a little more problematic than for platform types. Caisson construction also may be used for berthing and mooring dolphins, but its use generally is feasible only in large-scale offshore terminal facilities where more than one or two dolphins are required. At offshore terminals where water depths approach or exceed approximately 100 ft, prefabricated tubular steel structures like the ones evident in Figs. 1-1 and 1-2 are typically used. Prestressed concrete cylinder piles also may be used in deepwater applications. Another somewhat novel dolphin construction applicable to deep water and remote sites is the use of suction caissons to anchor the dolphin superstructure to the bottom, as described by Zhang et al. (2013). A suction caisson consists of a thin-walled steel cylinder capped by a lid at the top and allowed to sink into the bottom under its own weight with added ballast if necessary and then pushed to depth in the soil by external hydrostatic pressure as water is pumped out of the steel cylinder.

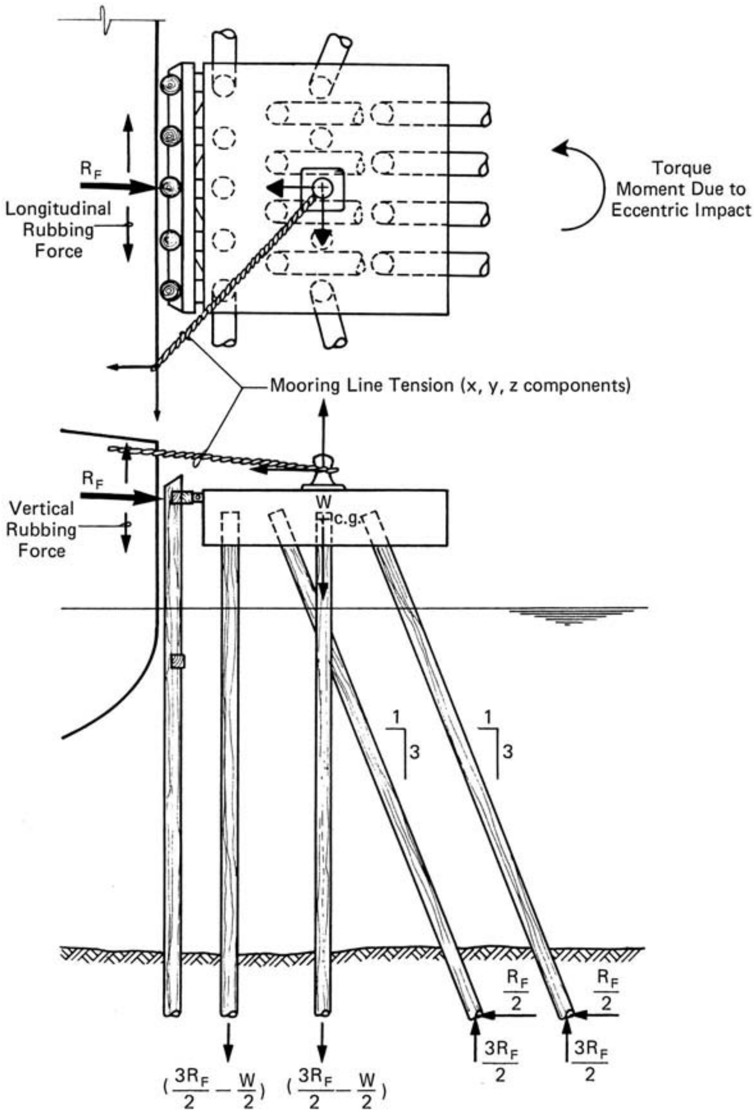


Fig. 7-26. Berthing and mooring dolphin load summary sketch

Two or more dolphins may be used to rigidly moor floating dry docks or floating pier structures. Floating dock systems for small craft commonly are moored by a series of cantilevered guide piles. Where the floating structure to be moored can be considered as a rigid body, the distribution of loads to the individual dolphins or guide piles can be calculated from basic static principles. Where only two dolphins are used, it usually is considered that at least two-thirds of the total lateral force is taken by either dolphin for preliminary design purposes. For the general case of

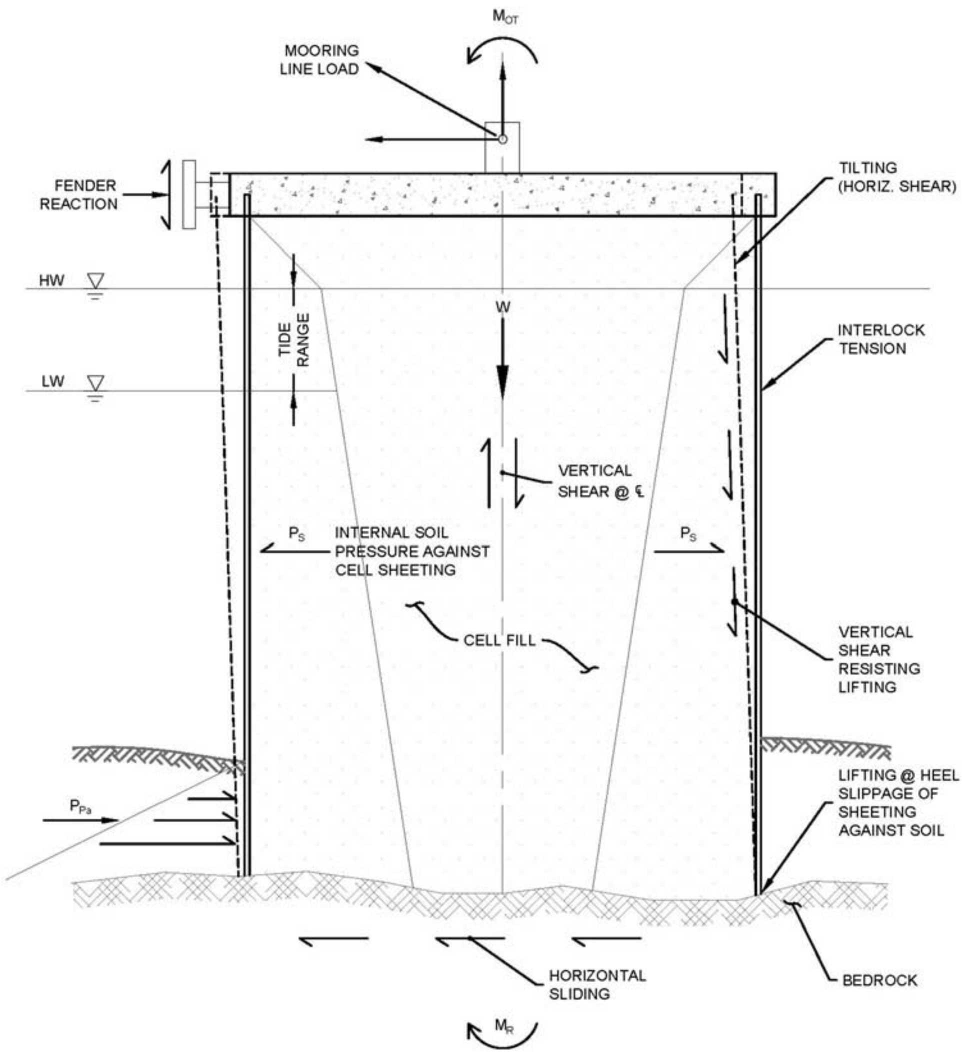


Fig. 7-27. Sheet pile cell dolphin schematic showing forces and soil reactions

multiple dolphins of varying lateral stiffness, the total lateral force (F_t) is divided among the dolphins in proportion to their stiffness, as found from

$$F_n = F_t \frac{n(EI)}{\sum EI} \tag{7-16}$$

where

F_n = load taken by the nth dolphin with stiffness (EI), and
$\sum EI$ = sum of the stiffnesses of all of the group.

It has been recommended that the force so found be increased by a factor of 1.3 for design purposes to account for possible misalignment of the dolphins (Tsinker 1986).

Service Platforms

Oil and bulk cargo terminals with breasting dolphin-type berth arrangements normally have platform structures located between the breasting dolphins that support hose towers or loading arms or other cargo transfer and conveyance equipment. Fig. 7-28 shows a combination liquid and dry bulk handling facility with hose towers and conveyors located on a central platform and hoppers located on sheet pile cell breasting dolphins. The service platform is usually set back from the berth face to avoid contact with the berthing and moored ship, although it may be fitted with some form of fendering to allow berthing of smaller craft such as tugs and barges and to protect against inadvertent impacts. The design of such platforms is similar to that of dolphin platforms, except that lateral loads from berthing vessels usually do not control. Lateral loads from extreme wind on elevated towers and conveyor equipment, however, may be significant. Equipment dead load and live loads must be provided by the manufacturer and/or terminal operator, and allowable movements under operational conditions must also be considered.

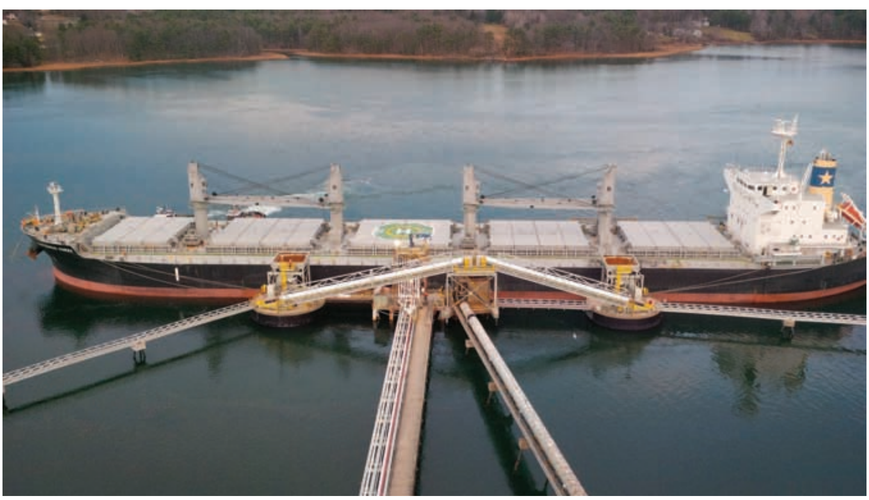


Fig. 7-28. Service platform at combination liquid/bulk handling facility with approach trestle supporting product lines and roadway and catwalks to access mooring dolphins

Source: Photo courtesy of Appledore Marine Engineering, LLC

Fixed Aids to Navigation Structures

Aids to navigation (ATONs) are common sights in ports and harbors, and although their design and installation are the responsibility of the U.S. Coast Guard, fixed aids often resemble dolphin structures, and port engineers should be familiar with their general characteristics. Design is often carried out by the regional U.S. Coast Guard Civil Engineering Unit but may be contracted to architectural and engineering firms, especially when major structures are required. ATON structures support visual and audible signal equipment at a design elevation that establishes the geographical range of the aid. Major structures include range towers and large offshore lights in deep water at typically exposed sites. Minor ATONs are generally located in shallower water or upon above-water obstructions, typically at protected to semiprotected locations, and are of simple construction, usually of wood, steel, or concrete piles, and may or may not be lighted. ATON structures in general provide support for a platform to allow access for service crews, dayboards, lanterns, sound signals, radar reflectors, solar panels, and battery boxes and are fitted with ladders and guardrails as required. Some ATON structures may also support prefabricated "skeleton" towers that elevate lights to the required height for the design geographic range. Minor ATONs, for example, typically have focal plane heights of 15 to 17 ft above mean higher high water (MHHW). Live loads include the weight of servicing personnel and their gear and lateral forces from alongside vessels and environmental loads from wind, waves, currents, and sometimes ice. Major structures are generally designed for a 50-year storm event, and minor structures may be designed for 10-year events, assuming that they will need periodic replacement. ATON structures may be simple monopiles, where subsurface soil conditions permit, or multipile groups of three or more battered or vertical piles, which may or may not require cross bracing. Fig. 7-29 shows a steel pipe pile tripod ATON structure, which are often used where monopiles are not feasible. The design of a tripod range tower is described by Elwood and Lund 2003, and the design of an offshore tripod structure controlled by wave forces is described by Gaythwaite and Mellor (2007). Steel pipe pile structures resembling small offshore platforms such as those used by the petroleum industry are commonly used in deepwater exposed locations, and their design is often controlled by waves and environmental forces for which the API Recommended Practice RP-2A (API 2014) provides design guidance. The general design of ATON structures is addressed in USCG (2005), and aids to navigation in general are addressed in the UFC 4-150-06 (DOD 2001b). Information and international requirements for marine aids to navigation can be found in the literature of the International Association of Lighthouse Authorities (IALA) (see Appendix 3 for website address).

7.6 Access and Ancillary Structures

Dolphins and platform structures are normally accessed by trestles, catwalks, or access bridges. Trestles are essentially bridges that are often of similar construction

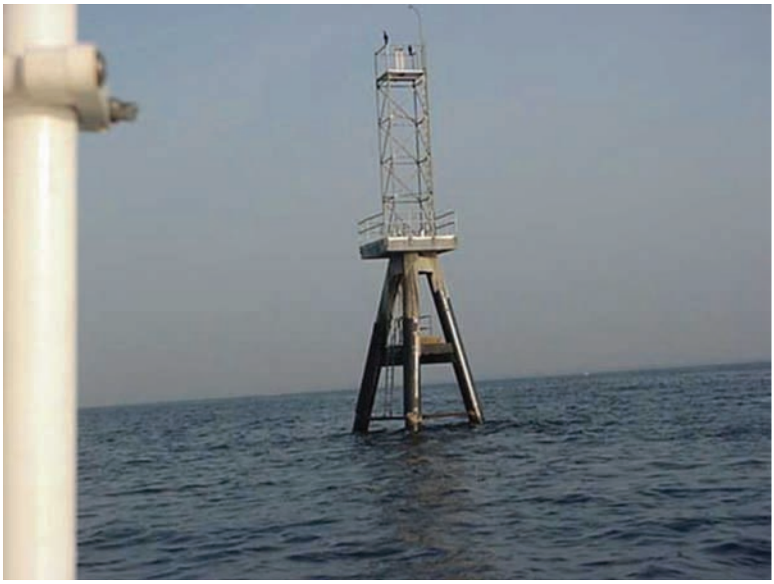


Fig. 7-29. Steel pile tripod ATON structure supporting a range tower with platforms, lights, and solar panels

Source: Photo courtesy of Appledore Marine Engineering, LLC

to the piers or platforms that they connect. Catwalks and utility bridges are usually of prefabricated steel open-truss construction with open-grating deck surface. Gangways and catwalks may alternatively be constructed of aluminum of the 5000 or 6000 series and designed in accordance with the *Aluminum Design Manual* by the Aluminum Association (AA 2010). Fiber-reinforced plastic (FRP) composite construction may also be appropriate for some pedestrian bridge applications (AASHTO 2008). Access bridges and catwalks may be of open, through-truss construction cross-connected only at the deck level, or they may be rectangular or triangular in cross section, as shown in Fig. 7-30, such that the main carrying trusses are laterally supported at the top and the bottom. The top chords of through-truss-type construction must be checked for lateral out-of-plane buckling stability. Tubular members often are used on longer spans and to carry heavier loads, such as utility systems. Long-span bridges should be constructed with an appropriate precamber to compensate for the structure and other permanent dead loads. A critical aspect of the design of all access structures is their means of end connection and ability to tolerate the maximum expected differential movements between the structures being bridged. Sliding bearings almost always are required at one end of the catwalk or bridge. Elastomeric bearings frequently are used and must be properly designed and installed to accommodate the expected loads, displacements, and rotations at the bridge ends (Husain 1983). In addition, access structures should be set back from the berthing face to avoid being struck by vessels, and the bottoms should be high enough above water to avoid wave action or their being struck by waterborne ice

Fig. 7-30. Steel catwalk with triangular pipe truss for dolphin access

Source: Photo courtesy of Appledore Marine Engineering, LLC

or flotsam. Trestles and bridges may also support product lines and utilities that may be subject to large thermal movements.

Catwalks intended solely for personnel access usually are designed for uniform live loads of from 40 lb/ft^2 for widths less than 3.5 ft to 100 lb/ft^2 for widths up to 6 ft. Catwalk spans normally are limited to around 100 ft for practical reasons, although much longer spans have been built. For narrow walkway structures, spans may be limited by the lateral stiffness of the deck. Deflections must be checked to avoid springiness underfoot and vibration problems. Lateral wind loads on catwalks can be significant and must be accommodated in the end connection design. The AASHTO (1997, 2009a) guidelines for design of pedestrian bridges provide general guidance and specify a uniform live load of 85 lb/ft^2 with a reduction factor for deck influence area but not less than 65 lb/ft^2. They also specify that the natural frequency be greater than 3 Hz to avoid the fundamental frequency of human traffic.

Trestles must be designed for all vehicular traffic or product lines to be accommodated (Fig. 7-28). Trestles usually consist of relatively short spans over independent pile bents and lend themselves readily to precast concrete highway plank construction. *Moles* are essentially solid-fill roadways; they usually are most economical in shallow water less than 10 to 15 ft deep and are often used at the shore end of access trestles.

Other ancillary structures include vessel boarding platforms and gangway structures called *brows* or pier stands. NAVFAC (DOD 2005a) requires that brows and brow platforms be designed for a uniform live load of 75 lb/ft^2 and be load tested with a 150-lb/ft^2 live load. Brow platforms may be stationary or mounted on caster wheels and may have several platform levels to accommodate large tide changes and vessel boarding locations. Boarding platform and gangway structures are available in a variety of sizes and configurations of prefabricated aluminum or steel. (See Section 9.6 for additional discussion of gangway-type structures.) Handrails should be provided, as described in Section 7.7. The Americans with Disabilities Act (ADA 1990) provides guidelines that may apply to many if not most facilities. The guidelines include maximum allowable slopes, lengths of inclines between landings, and minimum widths and clearances that must be complied with. Gangway slopes are limited to 1:12, for example, which can result in considerable superstructure and space requirements in areas with large tide ranges.

The design of transfer bridges for accessing floating pontoons or ferries and roll-on/roll-off (Ro/Ro) vessels directly from a pier or quay is addressed in Section 9.6. At facilities where the ramps from Ro/Ro vessels land directly on the shore structure, the edge of the pier, wharf, or quay must be properly configured as a fixed ramp to accept the ships' ramps and transfer vehicles while maintaining acceptable grades and ramp intersection angles. Design guidance for this requirement is presented in the British Standard BS 6349-8 (BSI 2007) and the ISO 6812 (ISO 1983). In general, fixed ramps are feasible where normal water levels do not exceed about 1.5 m. For larger water level variations, an adjustable ramp may need to be designed, which should be operational between 1.75 m above normal low-water level and 1.5 m above normal high-water level (ISO 1983). Bollards and any other pier edge appurtenances may need to be recessed below deck level in order to maintain a flush deck working area for the ships' ramps. Floating transition ramp structures known as "link spans" (see Section 9.1) may be used at some sites, and the pier or quay in turn must be designed to accommodate the moored link span.

7.7 Crane Trackage

Crane trackage and railroad trackage are common features on many piers and wharves. The design requirements of railroad trackage are covered in AREMA (2014) and DOD (2004), and their layout on waterfront structures as well as of portal crane trackage is discussed in DOD (1992). The following discussion applies to the general design of dockside cargo handling, portal and container cranes, and shipyard cranes, as well as shiploaders and rail-mounted equipment, as described in Section 4.3. Crane trackage generally consists of the following major elements:

- Rail and its mounting;
- Beam and foundation, including lateral support system for shiploaders and shipyard cranes without solid decks; and
- Ancillary features, including end bumpers, tie-downs, power feed details, and drainage.

Crane rails are made of heat-treated carbon steel in a variety of cross sections. Rails normally are designated by their weight in pounds per yard, followed by CR. In the United States, crane rails are available in sizes ranging from 40-CR to 175-CR, although the 104-CR is the lightest rail usually acceptable for waterfront applications. European rails typically have broader, flatter heads and a lower overall height and are available in sizes to just over 200 lb/yard. The maximum load a rail can carry is a function of the effective width of the railhead. Steel wheels generally range from 12 to 36 in. in diameter with maximum load capacities on the order of 35,000 to 176,000 lb. Of U.S. rail sections, the 171-CR is often preferred for heavy-duty crane service because of its relatively broad, flat head. Depending upon the class of service, this rail can carry wheel loads up to and exceeding 176,000 lb. CMAA (1994), Molyneux and Molyneux (1978), and Mazurkiewicz (1980) contain additional information on crane rail selection and installation requirements. Standard U.S. rail lengths are 33 and 39 ft, and larger rails are available in 60-ft lengths.

Rails may be provided with joint bars and predrilled holes at their ends. For dockside cranes, however, the use of continuous, electrically welded rails is strongly recommended. Rails can be supplied with prebeveled ends for welding on request. Crane rails usually are mounted on a continuous steel plate, generally between 3/4 in. and 1 1/4 in. thick, with bolted rail clips that allow the rail to slide longitudinally. The rail plate, in turn, is secured to the foundation with anchor bolts, usually 1 to 1 1/2 in. in diameter spaced at 2.5 to 3.0 ft. The rail plate is leveled and set in a bed of nonshrink epoxy grout. An alternative to the plate and rail clip arrangement is the use of closely spaced cast steel chairs, also set into an epoxy grout. Because of high local bearing pressures that can be developed and difficulty in achieving a perfectly uniform bearing surface, fabric-impregnated elastomeric bearing pads or steel-reinforced pads, usually ranging from 1/4 in. to 3/8 in. thick, should be placed below the rail. It has been noted that a heavy, moving, concentrated load causes the crane rail to bend, forming a “wave” ahead of the moving rail (Jenkins 1980). This wave action may result in high-tension loads in rail clips and anchor bolts. The use of fabric pad washers below the rail clip nuts may help to reduce peak uplift loads. Fig. 7-31 shows the installation of a crane rail for a large shipyard revolving gantry crane. Note the full-width fabric/elastomeric bearing pad below the base of the rail. Crane rails on piers and wharves normally need to be mounted with the top of the rail flush with the deck surface. Exceptions might be shiploader runways and shipyard applications where vehicle access is restricted. Pier and wharf crane rail mounting details are illustrated in DOD (2005a) and EAU (2004) and in the product literature of crane rail and hardware manufacturers.

Crane beams and foundation piles must generally be analyzed as a beam on elastic foundation. Although there is some distribution of peak wheel loads through the rail, wheel loads should be applied as concentrated loads for the analysis of the beam on elastic foundation. It generally is found that the rail beam bends over a long distance over the length of the crane bogie or series of wheels, rather than between the pile supports (Fig. 7-32). Crane beams must be designed for adequate stiffness, and

Fig. 7-31. Installation of crane rail for shipyard gantry crane before grouting below rail plate. Note anchor bolts, rail clips, and bearing pad

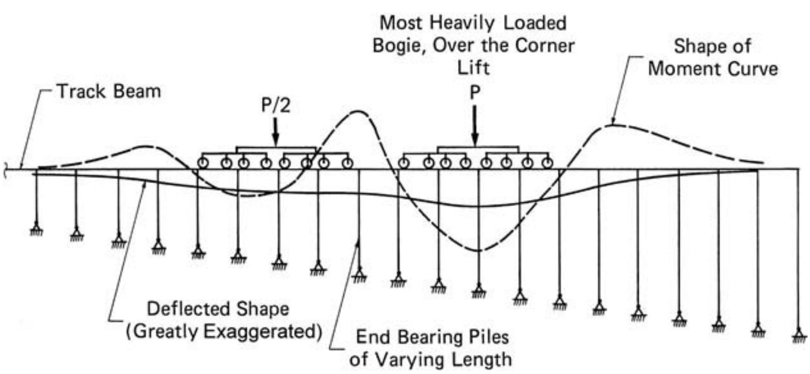


Fig. 7-32. Schematic of crane trackage moment and deflection variation below traveling bogie. Piles act as springs of varying axial stiffness

deflections should be as near to zero as possible. POLB (2012) requires that container crane rails be designed for a vertical load of 50 kips/ft with a load factor of 1.3 and a pile soil capacity factor of safety of 2.0 under normal operation. The weight of the crane itself is essentially considered as dead load. POLB (2012) further requires that the waterside crane beams of wharves consider two missing interior piles in the design analysis with a vertical load of 20 kips/ft and reduced load factor and that both waterside and landside rails be designed for lateral load of 3 kips/ft applied at the top of the rail. Operational wheel loads usually are given by the crane manufacturer for maximum traveling. For over-the-side and over-the-corner lifting conditions, refer to Section 4.3. There may be as many as eight wheels per crane leg, and spacings are usually between 2.5 and 5 ft. A 20% impact factor should be applied to all crane lifting loads. Individual wheel loads of container cranes often exceed 125 kips.

The beam and foundation should be analyzed for the following conditions:

1. Maximum crane operating loads,
2. Maximum operating load plus moderate wind (usually on the order of 25 mph), and
3. Maximum storm wind (crane secured).

Stresses for Condition 1 should be within nominal allowable working stresses. Increases in allowable stresses for Conditions 2 and 3 sometimes are taken as 15% and 33%, respectively. Lateral forces arise from wind, earthquake, crane slewing, and crabbing. Longitudinal forces arise from wind, earthquake, and braking and traction forces. According to NAVFAC (DOD 2005a), braking loads should be taken as 15% of the live load without impact and traction loads as 25% of the weight on the driving wheels on any one track without impact. Wind areas and vertical center of gravity and equipment weights required for wind and earthquake analysis must be obtained from the crane manufacturer, as should travel speeds and accelerations. Soderberg et al. (2010) review extreme loads on crane girders caused by wind, earthquake, missing piles, and crane mishaps. End bumpers must be designed to stop the crane when traveling at its full speed. BSI (2013) recommends a design velocity of 3.3 ft/s. Another design approach is to take the tipping moment of the crane as an upper bound, as in a runaway crane incident where the bumper force forms a couple with the inertial force acting at the vertical center of gravity of the crane balanced against the vertical load of the crane acting through its center gravity and the horizontal force per leg can be taken as approximately $0.25 \times$ vertical force (Bhimani and Soderberg 2006). Bumper loads should be obtained from the crane manufacturer whenever possible. Although the ends of the crane bogies often are equipped with cushioning devices, some resiliency should be built into the end bumpers as well.

It is generally desirable for crane rails to be recessed to allow passage of vehicles on deck. The crane rail pocket detail must allow for access to the rail clip bolts and must have some provision for drainage if not filled with removable filler material such as bituminous coal. Cranes are secured when not in use or during high winds, usually by means of clamping devices that grip the rail or sometimes by special tie-down arrangements. Pockets may be provided at desired tie-down locations to allow use of the crane's clamping device. Contemporary dockside cranes are powered almost exclusively by electricity. Most traveling gantry cranes are equipped with a self-spooling power cable near one leg, which emerges from the pier deck near the crane's center of travel. Sometimes, a recessed culvert may be provided for the power cable to rest in. Container and bulk handling cranes sometimes are fed power from elevated continuous trolley bus bars.

Crane rails should be located so that there is a minimum of 4 to 5 ft of clearance from the edge of the bogies to the pier face to allow safe passage of personnel for line handling. NAVFAC (DOD 2005a) recommends 7.5 ft minimum from the centerline of the rail to the pier face. Additional description of crane rail design and installation requirements can be found in CMAA (1994) and Molyneux and Molyneux (1978).

7.8 Miscellaneous Design Features

This section addresses important design features common to almost all pier and wharf construction. Fendering and mooring hardware are covered in Chapters 5 and 6, respectively.

Expansion Joints

Piers and wharves are subject to thermal movements and stresses associated with the extreme rise and fall of the structure's temperature about its normal mean. Because of the flexibility of pile bents normal to their plane, piers may be constructed over long distances without expansion joints, provided that longitudinal restraint is provided at only one location. Piers have been constructed to more than 800 ft without expansion joints, and [DOD (2005a)] allows distances of up to 600 ft. At locations where there is little seasonal temperature change, such as in the tropics, joint spacing may be even longer or expansion joints may be entirely omitted. Even when expansion joints are not provided, thermal effects and structure restraint must be investigated for concrete and steel structures.

The coefficients of thermal expansion for concrete and steel are almost equal and are usually taken as 0.0000065 units per unit length per degree Fahrenheit. The amount of temperature rise and fall about the normal mean temperature to be assumed for design varies with the local climate and with structural materials. For steel structures, the amount of rise or fall typically ranges from around 40°F for moderate climates to around 60°F in extreme climates. For concrete structures, the amount of rise or fall typically ranges from around 30°F in moderate climates to 40°F or 45°F in extreme climates. Thermal expansion effects normally are not considered for timber structures. In general, thermal extremes in waterfront structures are somewhat more moderate than for most land-based structures, although thermal gradients from deck surfaces in direct sunlight to deck undersides can be significant. For further guidance, see AASHTO (2014). Thermal expansion and contraction of large hydrocarbon pipelines can generate substantial lateral forces at fixed locations, depending upon the degree of fixity, even when

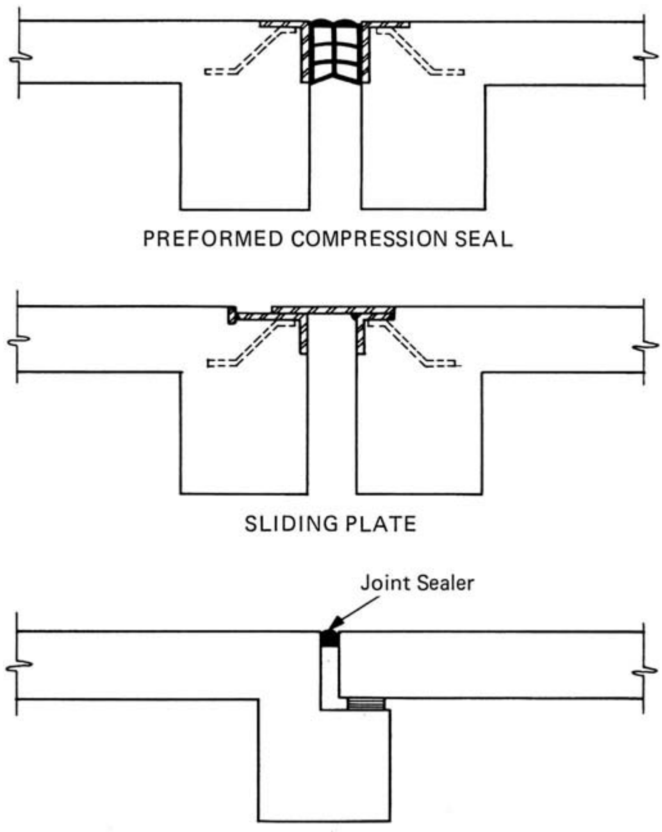


Fig. 7-33. Typical pier expansion joint details

expansion loops have been installed in the pipeline, especially for packed pipelines with large temperature variations.

Expansion joint gap widths must be properly set for the ambient temperature at the time of construction. The possibility of relative lateral displacements between adjoining pier sections should be investigated, and lateral shear keys should be provided as required. Care must be taken in detailing joints to prevent them from becoming clogged with debris. Preformed rubber units permanently bonded to galvanized steel angles cast into the concrete deck are a common solution. Another simple solution is the use of a sliding steel plate secured to a cast-in-place angle along one edge and allowed to slide atop a cast-in-place angle along the other edge. Fig. 7-33 illustrates some typical expansion joint details. Sufficient longitudinal restraint must be provided within each pier section between joints; otherwise, adjacent units may take a permanent set, effectively closing one joint and leaving too wide of a gap at the opposite end. Utility or product lines must be provided with suitable means, such as sliding sleeve arrangements, to accommodate relative movements at the joint.

Continuous welded crane rails should be allowed to remain continuous across expansion joints, but the anchored rail plate must not span the joint. Sufficient space at the ends of continuous rails must be allowed for their overall expansion.

Deck Drainage

Decks must be adequately drained with regard to local rainfall intensities or, in some cases, wave overwash. Deck drain openings may also provide an important venting function to wave uplift or air compression pressure, especially below open pile wharves. In general, decks should be pitched a minimum of 1/16 in. per foot toward drains and/or scuppers (curb openings) along the pier face. The down pipe should be at least 4 in. in diameter or an equivalent size, and a grate inlet should be provided to prevent the pipe from clogging with debris. Drains normally should be provided between every pile bent, or they should be otherwise sized and spaced to suit local runoff conditions. Crane rail pockets and slots should be adequately drained, usually with their own smaller, 1 1/2-in.-diameter pipe drains. Deck drain hardware is usually of cast iron or cast steel construction, consisting of a main frame, grate, and drainpipe. In some cases, polyvinyl chloride (PVC) may also be suitable for drainpipes and deck scuppers. Frames should be adequately proportioned to reinforce the opening in the concrete deck, and grates and frames should be heavy-duty and load-rated to suit traffic conditions. At petroleum oil loading (POL) facilities and at certain other facilities, interceptor and collection systems may be required to collect deck drainage runoff and conduct it to an oily water waste treatment facility. In general, deck surfaces should be as nonskid as feasible. Concrete decks can be broom finished, and for other materials ASTM C1028 determines coefficients of friction for nonskid surfaces.

Fire Holes and Stops

Piers and wharves with combustible substructures, such as all-timber and concrete decks on timber piles, should be fitted with framed fire hole openings to allow insertion of firefighting “fog” nozzles from above. Fire holes are usually on the order of 6- to 12-in.-diameter clear openings and may be fitted with grated covers to double as deck drains. Fire hole deck drains are not recommended at oil terminals because of possible spill hazard. Their spacing and location vary with the arrangement of the substructure and the local fire marshal’s requirements. All-timber structures also should have fire-stop cutoff walls extending from below deck to mean low water (MLW). The spacing, location, and details of fire-stops and deck openings should conform to the requirements of the National Fire Protection Association (NFPA 2000).

Curbs and Handrails

As a general rule, all piers and wharves where vehicular traffic is allowed should be fitted with continuous curbs along their waterside faces and edges. One exception may be Ro/Ro facilities that must accommodate vessels using their own transfer ramps. In reinforced concrete piers, a typical curb is on the order of 12 in. high by a minimum of 10 in. wide. Curbs, sometimes referred to as bull rails, may be made wider at bollard locations so that mooring hardware can be mounted directly atop them. Sometimes, the curb is discontinuous, and the mooring hardware is mounted directly to the deck slab. At some facilities where it is not possible to prevent mooring lines from leading down across the curb, chafing plates may be installed along the curb's outboard edge. *Chafe plates* often consist of a quarter round of galvanized steel pipe with anchor bars or studs cast into the concrete curb. Timber curbs may be used on concrete decks as well as on all-timber piers. They usually consist of 10×12 or 8×12 treated timbers, which are elevated slightly above the deck by chocks to facilitate drainage and prevent the onset of timber decay. Curbs may be omitted entirely at passenger-handling facilities where handrails are required. Handrails should be a minimum of 3 ft 6 in. high from the deck to the top rail, and preferably have three rails. At passenger terminals, handrails must conform to local building code requirements. Webbing or wire mesh may be installed between rails and stanchion posts for passenger and child safety. Steel pipe or tube rails should be hot-dipped galvanized, and the top rail should be of a larger dimension than the lower ones to provide a comfortable hand grip. Removable sections of rail often must be provided. Vertical stanchions with hooks for chains in place of rails may be used at such locations. Rails and their anchorage to the pier deck should be designed to withstand a continuous horizontal and vertical force along each rail of 50 lb/ft, or as otherwise required by highway bridge design standards for combined traffic loading.

Ladders

Ladders often are provided to facilitate access from small craft and for safety reasons. POLB (2012) requires ladders to be located along the pier face no more than 400 ft apart. The ladder should extend downward to approximately 1 to 1 1/2 ft above MLW for boarding from small craft. Ladders that are provided primarily for "man-overboard" safety reasons preferably should extend 2 to 3 ft below MLW to facilitate climbing out of the water at low tide. However, the lower portions are then subject to fouling and corrosion. Barnacles and other fouling growth can make ladders treacherous. At some locations, the use of ladders is discouraged for security reasons; thus, the lengths and locations of ladders vary with facility types and owner/operator and OSHA requirements. Ladders should have handrails at the deck level to help users pull themselves on deck, and they should be recessed flush with the dock face or be sufficiently set back from the fender face to preclude damage. Also, ladders must be adequately secured against sway in all directions, and the design of rungs should consider the impact of the concentrated weight of a large person carrying gear. Rungs are usually from 1.5 to 2.0 ft wide, spaced at approximately 12 in. Rungs of wooden ladders must be let into the verticals, and in no case should metal fastenings be relied upon as the sole vertical support. Rubber ladders are now available and are rapidly becoming a worldwide standard to provide emergency egress from the water.

Lighting and Light Poles

Pier lighting is required for safety, security, and operations. Topside deck lighting is most commonly provided, but under-deck and even underwater lighting may be required for security reasons at certain facilities, such as military bases. DOD (2005a) calls for an average of 5 foot-candles illumination in active work areas down to 0.5 foot-candles in other areas, such as entrances, foot traffic areas, and ship berth area out to 600 ft from the pier face. The minimum number of high mast poles that provide uniform coverage should be provided. Fixtures should in general be of metal-halide or high-pressure sodium with full cutoff or fully shielded luminaires to reduce glare and not blind vessel operators. Light pole hazard area restrictions may apply various lighting systems and distances to transfer points at marine oil terminals. For waterfront lighting design guidance, see DOD (2015), and for security lighting and port facility security requirements in general, see DOD (2012). Wind loading drag coefficients and fatigue criteria for anchorage connections of light poles and other signage support structures can be found in AASHTO (2009b).

Anchorage Details

Various hardware and equipment must be anchored to pier and wharf structures by anchor bolts, including fender systems, mooring hardware, pad eyes, davits and hoists, product lines, and services. Anchor bolts must be amply sized to secure the base of the fitting being anchored under all expected horizontal and vertical loads and overturning moments, and to develop the full-capacity safe working load (SWL) of any load-rated hardware. The anchor bolts may be made of mild steel for various appurtenances; however, high-strength steel bolts or anchor rods are usually specified under ASTM F1554, *Standard Specification for Anchor Bolts, Steel, 36, 55 and 105 ksi Yield Strength,* usually Grade 105, for the anchorage of critical items such as mooring bollards and quick-release hooks. Anchor bolts should normally be hot-dipped galvanized or of stainless steel at highly corrosive sites. It is desirable, wherever possible, especially for critical hardware, that bolts be through-bolted in pipe sleeves to allow for their eventual inspection and replacement. Critical mooring hardware should be fitted with backing bars between adjacent bolts. Anchor bolt connections on timber pier construction should be solidly chocked. An ample corrosion allowance should be provided for anchor bolts, and an additional protective coating, such as tar pitch, should be applied to the nuts and threads after tightening. Nuts should be properly torqued in accordance with AISC (2010) specifications and installation guidance given in ACI (1994). Grouting below hardware and equipment bases is covered in ACI (2014c, 2015). Additional information on minimum hardware sizes and NAVFAC general design requirements can be found in DOD (2001).

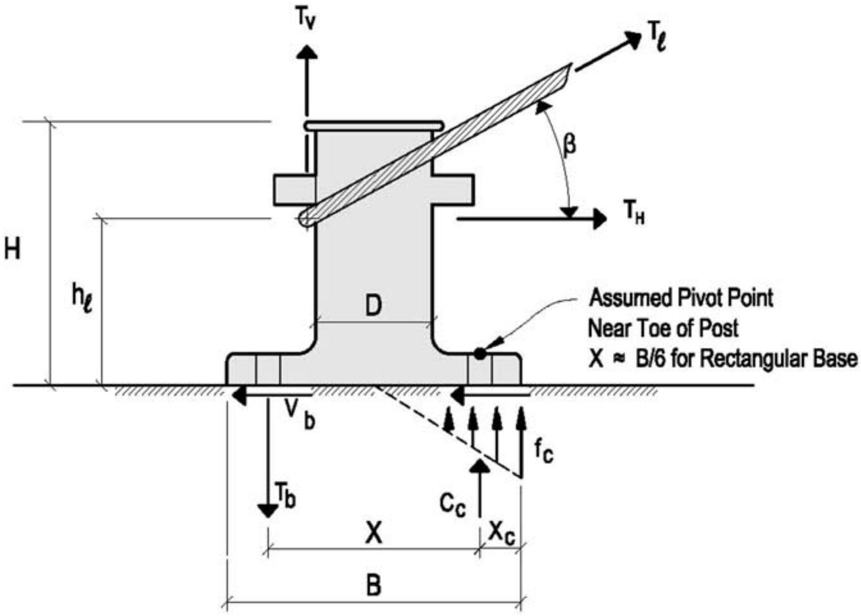


Fig. 7-34. Bollard anchorage schematic showing forces and reactions

Fig. 7-34 illustrates the forces acting on a typical bollard installation. Loads on mooring hardware are addressed in Section 6.4. These loads can result in substantial anchor bolt loads on bollards and quick-release hooks. The mooring line tension exerts an upward pull and overturning moment in addition to the horizontal line pull that put the anchor bolts in combined shear and tension. A point of rotation near the base of the bollard post may be assumed generally around one sixth of the base width from the edge for a rectangular base in order to determine the anchor bolt uplift. Various directions of line pull in three dimensions must be considered to find the maximum bolt load condition. In many instances of new designs, the shear can be eliminated by recessing the bollard base a sufficient amount into the concrete deck. Additional local reinforcement is often required to meet ACI 318 (ACI 2014a) anchorage requirements because mooring hardware is typically located near the edges of concrete deck systems. Through-bolting can help mitigate this problem, but where postinstalled concrete anchors are required, this can become somewhat problematic, especially when securing new hardware to existing decks when the existing deck reinforcing is unknown.

Markings and Navigation Lighting

Pier decks are often painted with yellow safety lines to delineate the outline of crane bogie travel or the swing radius of rotating equipment. Berth identification numbers also may be painted on the dock face or inside face of the curb. Such marks are designated by the owner/operator of the facility. Navigation marks such as day marks, radar reflectors, fog signals, and lighting may be required at exposed berths, especially at offshore locations; these marks fall under the jurisdiction of the Coast Guard in the United States and usually must conform to accepted international standards. In general, a fixed or flashing amber or yellow light, which has no lateral significance, is used to define the extremities of a fixed or special-purpose structure. Navigation lights often have their own power supply and may have solar cells to keep their batteries charged. Navigation marks or aids to navigation (ATONs) for a specific project must be approved by the Coast Guard and local harbor authorities. Range lights may be used, usually at head-on berth approaches, to assist vessels docking at night and under reduced visibility.

7.9 Ship Services and Utility Systems

Marine facilities may be equipped with a wide variety of service and utility systems, depending upon the nature of the operations. This section is intended to review the various kinds of systems and their general requirements; the mechanical and electrical design of such systems is beyond the scope of this book. The most common dockside services usually provided are water, electric power, sanitary waste, and communications systems. Water is supplied to the vessel as potable water for drinking and washing, and the pier also may have a nonpotable water supply for washing down and flushing and for fire protection.

Fire protection systems may be saltwater supplied from the pier's own pumping station. Firefighting requirements vary and are subject to the approval of local fire officials. The U.S. Navy's requirements per UFC 4-150-02 (DOD 2003), for example, range from 1,500 gal./min for a single berth up to 2,000 ft long with a minimum residual pressure of 40 psi to a maximum of 3,500 gal./min with a minimum of 125-psi residual pressure at the most remote outlet for an unlimited number of berths. Hose connections are typically 2 1/2 in., spaced so that any one location can be reached from more than one outlet with a maximum hose length of 300 ft. In addition, the fire protection system should have 44-in. pumper connections. International shore connections should be provided for direct supply to the ships' fire mains.

Potable water requirements range from 1,000 gal./min at 40 psi minimum for a single berth up to 2,000 gal./min at 40 psi for multiple berths more than 2,000 ft long. The ship service system should be equipped with double-check backflow preventers; 2 1/2-in. connections taken from a 4-in. main usually are spaced at approximately 120-ft intervals. A good choice of materials for exterior dock waterlines is cement-lined ductile iron pipe with cellular glass insulation covered with either aluminum or stainless steel sheeting secured with stainless steel bands.

In freezing climates, waterlines must be protected by heat tracing or by being run inside utility trenches or enclosures with live steam lines. Water supply lines that are infrequently used should be self-draining to prevent freeze-ups. Saltwater systems are often kept "live" with small inductor pumps that continually circulate a small volume of water, which is discharged through a small orifice or valve at the end of the line. Sewage usually is handled with an 8-in. line with flexible hose connections from the ships. Sewage connections may be spaced up to 200 ft apart or, in some instances, at a single location for single berths.

Electrical power supply requirements range from a minimum 460 V/3 phases to 15 kV to operate container cranes and illuminate large marshaling yards. Military ships are normally connected to shore power when they are in the so-called cold irons condition. Commercial ships, in general, seldom require shore power connections. There is a trend, however, that cruise ships require shore power in order to avoid using the auxiliary machinery to generate power in environmentally sensitive locations, such as California and Alaska, and also in locations where emission of CO_2 is taxed, such as in Norway. "Cold ironing," however, has now become more common at certain facility types in the sister ports of Los Angeles and Long Beach, California, and is likely to become more widespread, especially at berths that are frequented by cruise ships or military ships staying for protracted periods at the berth. It is essential that all electrical services be properly grounded in accordance with the National Electrical Code requirements. Dockside transformer frames and metallic receptacle enclosures should be grounded through an equipment-grounding conductor. Electric conduits, usually 5-in.-diameter PVC pipes, are often cast in place inside the beams of concrete piers, direct buried, or in duct banks suspended under the pier. Exposed raceways, either above grade or suspended under piers, should be vinyl-coated rigid galvanized steel conduit and fittings. Telephones or voice-activated sound-powered communications often are provided at locations where electrical and other utilities are grouped together for ship connections. Sometimes, small shelters or covered boxes are provided at utility connection locations. Electrical distribution equipment housing receptacles, circuit breakers, switches, junction boxes, and so on should be fabricated from 316-type stainless steel conforming to the National Electrical Manufacturers Association, NEMA 4X construction. Although noncorrosive, fiberglass or polycarbonate enclosures tend to degrade with exposure to ultraviolet light. Mounting hardware (e.g., bolts, nuts, washers, and drive pins) should be Type 316 stainless steel. Pier deck lighting normally is provided by light standards ranging from 20 to 50 ft high. Standards up to 120 ft are provided for container wharves. For security only, a minimum illumination of 0.2 to 0.5 foot-candle should be provided. Nighttime cargo-loading operations require a minimum of 5 foot-candles, and passenger waiting areas require more than 20 foot-candles.

Steam lines may be provided for space heating and for warming utility trenches and are essential in shipyard operations. Steam conduits are usually carbon steel pipes. If space allows, maintenance-free expansion loops are preferred. If mechanical expansion devices (e.g., bellows of the piston type) are used, then sufficient space should be provided around them for maintenance. Compressed air may be supplied for operating equipment, especially in hazardous areas where electrical sparks present a safety hazard. In shipyards, various gases (such as oxygen, natural gas, argon, and acetylene) frequently are piped from the pier, and they have significant spatial requirements. Many shipyards also provide connections for the removal of oily ballast water from the vessels. Ballast water is often pumped into separating tanks set inside a secondary containment structure.

Bunkering or fueling of vessels usually is carried out from barges or oilers supplying bunker C or diesel fuel, so such services generally are not provided from dockside, except at ferry terminals.

Ship services and utility systems may be carried directly on the pier deck, although they are most commonly carried below the deck on pipe hangers or within cast-in-place utility trenches. Utility trenches may run along the inshore margin of a wharf with periodic crossovers to utility stations or bases; or there may be one crossover, with the trench running along the outshore face with risers at box locations. On piers with two-side berthing, the trench often runs down the middle of the pier with branches to both sides. Utility trenches must be designed for ready access to piping and must have adequate drainage, so the bottom of the trench must be kept above the highest tide levels. Where this is not possible, the trench structure must be watertight, and utility trench covers should be sealed to prevent water intrusion; other provisions must be made for keeping it dry inside. Manholes should have sumps with gravity or electric-powered pumps for drainage to the nearest storm drain. Fig. 7-35 illustrates some typical below-deck utility trench details. Dockside service systems can be

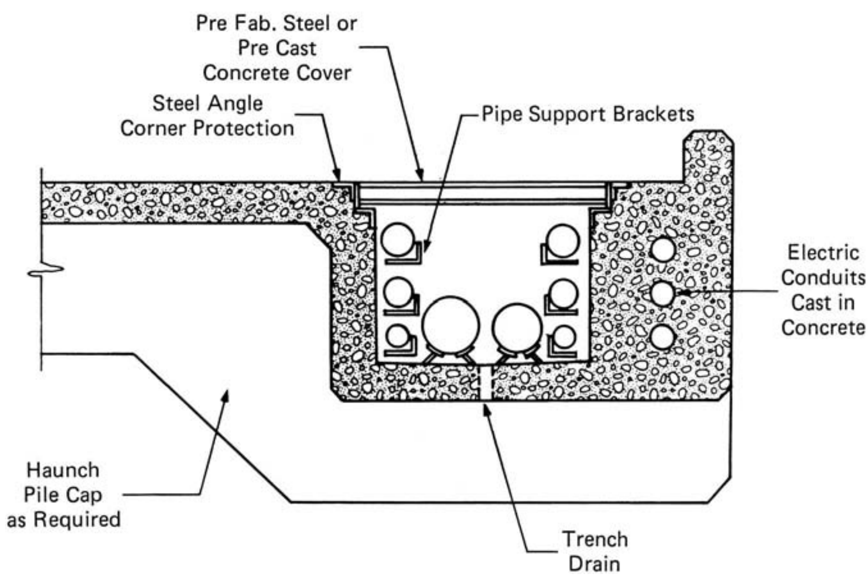


Fig. 7-35. Typical below-deck utility trench details

Fig. 7-36. Utility hood protecting risers and valves

Source: Photo courtesy of Appledore Marine Engineering, LLC

Fig. 7-37. Electrical services vault

Source: Photo courtesy of Appledore Marine Engineering, LLC

grouped into somewhat similar categories for risers, manifolds, and connection locations. Per DOD (2003):

- Freshwater, saltwater, steam, and air;
- Electrical and communications (must be isolated at least 10 ft from others);
- Sewer and oily wastes (must be isolated at least 10 ft from others); and
- Petroleum, oil, and lubricants (POL).

Exposed pipe risers, manifolds, and utility connections should be protected by parapet walls, bumping posts, enclosures, rope guards, or other suitable means (Figs. 7-36 and 7-37). More in-depth description of dockside utility systems can be found in DOD (2003).

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CHAPTER 8

Geotechnical Design Considerations

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The design of waterfront and nearshore structures is greatly dependent on site history, local geology, and geotechnical characteristics of site soils and bedrock. As in many foundation designs, structural members and natural materials combine to form a soil–structure interaction (SSI) design challenge. This is particularly evident in the design of marine structures such as flexible bulkheads, solid-fill piers and wharves, and graving docks. In these facilities, soil and rock not only provide a foundation for the structure, but they may also furnish resistance to overturning, uplift pressures, and stability failure. Ironically, while providing these benefits, the soil or rock mass also may be the source of the greatest structural loads.

The characteristics of the soil and rock at nearshore sites can vary widely in areal extent, as well as with depth, because of the dynamic environment in which they were deposited or reworked. These materials may be found in a natural undisturbed state or may have been altered by previous human activities. Consequently, the marine designer not only must be proficient at structural engineering and the determination of environmental loads, but also must be acutely aware of the many variables associated with geotechnical design in the coastal zone. An intimate knowledge of soil mechanics and foundation engineering principles is also essential, along with a firm understanding of site geotechnical characteristics, geology, and history.

This chapter first outlines site investigation and subsurface exploration techniques, material characteristics, and testing methods. A discussion of geotechnical design considerations for various types of waterfront and nearshore structures follows. Particular attention is given to the details that make geotechnical design in the coastal zone different from more traditional upland work. Finally, a discussion of site improvement techniques is presented to illustrate possible uses in the construction of waterfront structures.

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8.1 Marine Site Investigations

Previous development history, local and regional geologic and geotechnical characteristics, and coastal processes can have a profound effect on the design of waterfront and nearshore facilities. The structural type and construction costs may be greatly affected by these factors. Therefore, adequate site investigations are essential to the design of marine facilities.

A well-planned investigation program is fundamental to attaining sufficient data for design. Site investigations should be performed in three phases: (1) collection of existing data and historical information, (2) completion of a detailed site reconnaissance, and (3) implementation of a subsurface exploration and field-testing program.

Existing Data Collection

Existing site data and historic information can be obtained from several sources. Geologic, topographic, and coastal feature information is often available in the form of surficial geology, soil science, agricultural and topographic maps, navigation charts, or summary reports compiled by local or national geologic and oceanographic organizations. Additionally, recent advances in geographic information systems (GIS) at state and federal agencies have provided additional sources of existing data, including computer-based topographic, bathymetric, and regulatory information and photography. Hydrologic data and well logs also may be available through state and federal water resource departments. Municipal building departments may also be a source of historic drawings and plans of existing waterfront facilities.

Aerial photographs can be used to document the existence and location of previous structures, site filling history, and changes to the shoreline. Old surveys, site plans, construction documents for structures and utilities, and subsurface exploration information may be available through physical plant or facilities personnel at existing industrial or military facilities. Communications with long-term residents or employees of the facilities in question can lead to valuable information pertaining to site history, filling, former construction, and previous industrial practices.

When existing or former industrial sites are being studied, it is essential that the investigator determine previous site use, activities, and products manufactured, processed, or handled. Many cases warrant a full environmental site assessment and internal audits of buildings. The cost of removal and disposal of contaminated soils and hazardous materials may far outweigh the benefit of the project, and the selection of another site may be more appropriate. Environmental information pertaining to the site may be available through the state environmental agency; the local fire department, which keeps records of underground storage tanks and hazardous materials; fire insurance maps; or through one of the online environmental data search services.

Site Reconnaissance

A site reconnaissance should be performed after a thorough review of available information. The reconnaissance usually consists of a detailed walk-through and visual observation. General site characteristics, surface soils, rock outcrops, subsided or eroded areas, existing structures and foundations, or any other unusual topographic feature should be noted and located. The effect of coastal processes, littoral currents, and natural deposition and erosion might also be inferred. Existing data should be compared with previous information in order to determine changes to the site. Updated aerial, topographic, and bathymetric surveys may be warranted if existing information is not current or adequate.

Existing upland and waterfront structures should be investigated to evaluate their general condition, foundation type, and performance. If it is anticipated that these structures will be rehabilitated or reused, a much more extensive structural and materials condition survey should be performed (Gaythwaite 1984) (see Chapter 11). A determination of how existing bulkheads are anchored also is important and may have serious implications on future nearshore construction. Special attention should be given to the possibility that former seawalls and other marine structures may have been buried by subsequent land reclamation.

Subsurface Explorations

Subsurface explorations on land and in the nearshore environment are required to evaluate soil, rock, and groundwater conditions for the proposed development. Subsurface explorations generally consist of drilled borings and test pits and may include geophysical surveys. Borings and test pits allow for detailed observation of subsurface soils and rock while providing investigators the opportunity to obtain samples for laboratory testing, perform in situ tests, and install groundwater monitoring wells and equipment. Drilling and test pitting operations tend to be costly, however, and only supply information pertaining to a limited portion of the site. Geophysical methods allow for broad interpretation of large areas but lack the detailed information required for design. A combination of two or more methods may be the most efficient and cost-effective way to map site subsurface conditions. In addition to these methods, aerial and satellite photography may be useful in locating large-scale geologic features.

Drilled Explorations

The most widely used method of subsurface exploration is the boring of holes into the ground using rotary drilling, augering, percussion drilling, cable tool methods, or cased drive and wash techniques.

Powered rotary equipment can be mounted on trucks, tracks, all-terrain vehicles, small floats, barges, jack-up boats, and ships (Figs. 8-1 and 8-2). Shallow, nonrotary drilling can be accomplished in areas inaccessible to other rigs by using a small

Fig. 8-1. Jack-up barge for deep or rough water drilling

Source: Photo courtesy of GZA GeoEnvironmental, Inc.

tripod and a gas-operated cathead and pump. A probe-type rig that pushes a small-diameter, plastic-tube-lined casing into the ground can be used to cost-effectively obtain a continuous record of relatively shallow soft or loose soils. The methods and equipment used depend on the size and depth of the explorations, the type of sampling and field testing required, and the physical character and accessibility of the project site. Table 8-1 lists several types of drilling equipment with their common uses and capabilities.

A typical borehole is advanced by alternately drilling and sampling. Samples generally are taken at intervals of 5 ft, or less if changes in soil type are encountered, using a split-spoon sampler and the standard penetration test (SPT) methods (ASTM 2002). SPT data can be empirically related to soil engineering characteristics pertaining to foundation design (Bowles 2009). The recovered soil sample is used for visual and laboratory classification of soil types. Larger diameter undisturbed samples of cohesive soils can be obtained for laboratory testing using one of several types of thin-walled sampling devices (Hvorslev 1949). Where the quality and the character of bedrock are of concern, rotary-driven rock coring equipment is required to obtain samples (Hvorslev 1949).

In Situ Testing

In addition to the SPT, several types of downhole tests are available (Robertson 1986). In situ testing is used to determine design parameters that may be

Fig. 8-2. Typical float-mounted drill rig

Source: Photo courtesy of GZA GeoEnvironmental, Inc.

incorporated into a direct design approach. Certain types of in situ equipment can provide a continuous record of the subsurface profile. This record gives a good indication of site variability, and it can provide characterization of soils that normally are difficult or impossible to sample and test in an undisturbed state.

Typical methods of testing in situ strength or properties of soils include vane shear, flat dilatometer, and pressure meter. Installation of piezometers or monitoring wells can be used to determine piezometric levels at specific locations or the phreatic groundwater surface. Field-testing equipment and instrumentation also

Table 8-1. Subsurface Exploration Equipment

Exploration	Use	Capabilities	Comments
Hand-excavated test pit or auger hole	Sampling for visual identification and laboratory testing	Shallow explorations in soils above the water table	Provides data in inaccessible areas, limited depth of exploration
Backhoe (test pit excavation)	Bulk sampling and visual identification of soils, in situ testing, determination of groundwater and bedrock elevations	Possible depth range from 20 to 30 ft, but generally shallower below water in cohesionless soils	Access may be difficult, limited undisturbed sampling, difficult below water
Tripod boring rig	SPT sample using casing drive and wash techniques	Limiting depths range between 25 and 40 ft	Provides boring data in otherwise inaccessible areas
Truck-mounted drill rig	Deep drilling with SPT, cone penetration, undisturbed sampling, in situ testing, and well installation. Boring may be advanced using augering, rotary drilling, percussion drilling, cable tool methods, and casing drive and wash techniques	Possible depths to greater than several hundred feet	Limited access in some situations, efficient and cost-effective method of data collection
All-terrain vehicle or track-mounted drill rig	Similar to truck-mounted rig	Similar to truck-mounted rig	Provides drilling access to undeveloped sites; production somewhat slower than truck-mounted equipment
Float-mounted drill rig (anchored)	Water borings with SPT and undisturbed sampling using casing drive and wash techniques	Operation in 40 to 60 ft of water, drilling approximately 100 ft below mudline	Limited to protected sites, sizes and power of rig limited, susceptible to sea condition, limited rock coring

Barge-mounted drill rig (spud barge)	Water borings with SPT and undisturbed sampling using rotary drilling or drive and wash techniques	Operation in 30 to 40 ft of water, drilling to several hundred feet below mudline	Limited to relatively shallow water, somewhat less susceptible to sea condition than float-mounted rig
Barge-mounted drill rig (jack-up)	Similar to spud barge	Operation in up to 80 ft of water, drilling to several hundred feet below mudline	Provides table platform that avoids problems with sea condition and tides, can be fitted with large drilling equipment
Barge-mounted drill rig (anchored)	Similar to spud barge	Operation in more than 100 ft of water, drilling to several hundred feet below mudline	Can operate in relatively deep water, susceptible to sea condition, may require large anchoring equipment at exposed sites
Boat-, barge-, or jack-up-mounted vibratory corer	Over water unconsolidated sediment sampling—continuous cores up to 40 ft long	Operations in up to 150 ft of water and samples up to 20-ft in length are normal, though deeper water sampling, and longer cores are possible	Four to ten cores/day common, but samples should not be considered undisturbed. Ship platform common for 20-ft or shorter samples. Barge/crane generally required for 40-ft cores

Notes: SPT means standard penetration test. Core penetration testing (CPT) can be performed from a large barge or jack-up equipment.

may be required during construction to monitor subsurface soil and structure response and may include earth pressure cells, strain gauges, inclinometers, settlement gauges, piezometers, and cross hole sonic testing to determine seismic characteristics (Dunnicliff 1988).

Borehole logging is a downhole method that is applicable to marine-related projects because of the high cost of obtaining subsurface data offshore. A downhole geophysical probe is lowered to the bottom of the hole at the completion of the boring. The multiinstrument mounted array is raised at a constant rate to map borehole sidewall properties. Typical properties measured include resistivity, density, and shear wave velocity.

Table 8-2 lists several types of in situ testing and field-monitoring equipment commonly used during the exploration and construction phases of waterfront projects.

Environmental Testing

Following the existing data search for environmental information, samples for laboratory testing can be obtained. It is often cost-effective to combine the geotechnical and environmental explorations and sample gathering into a comprehensive field program. The same explorations can often be used for both studies. Explorations can be planned for areas of environmental concern, with additional explorations appropriately spaced for geotechnical issues. Explorations over water tend to have high mobilization costs. Therefore, it is even more important to limit the number of water-based exploration programs. Environmental samples for dredging should be taken when the offshore geotechnical explorations are performed. It is important to note that soil samples taken for environmental purposes require special handling protocols and may have limited time frames for testing. Careful planning for a combined geotechnical and environmental program, therefore, must be done proactively.

Cone Penetration

The cone penetration test (CPT) is a method of in situ testing for subsurface soil characterization in which an instrumented device with a conical tip is pushed into the ground with rods at a constant rate. The core on the end of the device has a base area of approximately 10 cm^2 and an approximate angle of 60° (Robertson 1989). The instrument measures tip resistance, shaft resistance, pore water pressure, shear wave velocity, and electrical conductance, among other characteristics. The CPT is most applicable to fine-grained and loose deposits. Its use in coarse and dense soils is limited because of the high tip resistance.

The main advantages of the CPT method are that a large volume of soil can be characterized efficiently and economically, the soil is tested continuously throughout the depth of the exploration, and the equipment measures in situ soil properties

Table 8-2. In Situ Testing and Monitoring Equipment

Device/Test	Description	Use	Comments
Standard penetration test (SPT)	Split-spoon sampler driven into soil at bottom of borehole in accordance with ASTM D1586	Determination of soil density and consistency and inferred soil properties based on sampler resistance (blow count)	A disturbed sample is retrieved for visual identification. Test results vary depending on equipment and operation
Cone penetration test (CPT)	A hardened steel, cone-tipped probe instrumented with load cells, pressure cells, and other site-specific devices. A continuous record of data is obtained as the probe is pushed into the ground	Development of a detailed stratigraphy profile, determination of ϕ angle, relative density, pore pressure, coefficient of consolidation, and other properties, depending on instruments used	Penetration into dense soil may be limited, depending on equipment used. The CPT has been extensively used, and many design procedures have been developed based directly on the results
Pressuremeter	A cylindrical membrane is inserted into a borehole. The membrane is inflated, while corresponding pressure and strain measurements are recorded	Determination of elastic moduli of soils and ultimate bearing capacity. Other properties may be inferred	Test is sensitive to borehole preparation. Test procedure is relatively simple. Data reduction and interpretation are completed using a computer program
Dilatometer	A thin blade fitted with an earth pressure cell on the face. The blade is pushed to some depth into the soil, where pressure measurements are taken	Determination of pore pressure, stress history, in situ stress, and other properties	A rugged device that may penetrate dense soils. Test may be run rapidly at each depth with little operator variability
Field vane	A four-bladed vane that is pushed into undisturbed soil to some depth and rotated. Resistance is measured until the in situ shear strength is exceeded	Determination of peak and residual undrained shear strengths	Test procedure is highly variable and may produce erratic results. It is impossible to determine direction of weak shear planes
Borehole shear device	A device that applies a normal force to the borehole walls and is pulled upward at a constant strain rate while the resistance is measured	Determination of shear strength, ϕ angle, and cohesion	Simple device. Tests may be run quickly. Results tend to be reproducible

directly. The main disadvantages are the relatively high CPT rig mobilization cost and limitations in coarse-grained and dense soils.

CPT devices can also use a variety of methods other than pushing. Remote CPT equipment can be used in standard cased test borings or with wireline equipment or on a bottom-deployed, remotely operated device. The remote device records and stores data until it is retrieved.

Vibratory Coring

Vibratory coring is a commonly used overwater method to obtain continuous samples up to 40 ft long of unconsolidated material. Samples are generally recovered within a clear polycarbonate liner that is then cut to 5-ft lengths for handling and transport to a laboratory. Vibratory coring or "vibro" samples are not considered undisturbed because of the vibratory action of the equipment, but they can be used for classification and environmental testing. Pneumatic and electric vibratory corers are common, whereas hydraulic-powered units are less common because of possible hydraulic oil spillage. Large pneumatic units are generally more powerful than the available larger electric-powered units and are thus able to recover longer samples and/or penetrate more compact sediments.

Test Pit Explorations

Test pit explorations are used extensively on land, and less commonly below the water, to supplement drilled explorations. They are useful in determining existing foundation types, locating buried seawalls and other waterfront structures, accessing subsurface structures for materials condition and structural surveys, characterizing the nature of fill or shallow natural soils, and exploring for environmental contamination. Test pit excavations are relatively inexpensive and can reach a practical depth of approximately 20 ft, depending on soil and water conditions. Test pit explorations do not have the obstruction limitations common to other exploratory methods.

Geophysical Methods

Several geophysical methods are available to the marine engineer to facilitate the characterization of subsurface conditions. Seismic reflection methods, depending on the operating frequency, can be used to develop profiles of the bathymetry, location of the bedrock surface, and location of distinctive soil horizons (USACE 1995). High-resolution multibeam bathymetric systems use reflected acoustic pulses, as do their single-beam counterparts, and have become smaller and less costly, allowing cost-effective high-density coverage of entire underwater bottom areas. Seismic reflection, in conjunction with limited borings or vibratory cores, is of particular use in determining types and qualities of material for channel and berth dredging and the elevation of the top of a hard soil layer of bedrock for pile-driving considerations. Side-scan sonar can be used to locate and characterize bottom features, bedrock, outcrops, and sea floor obstructions. When coupled with a magnetometer survey, side-scan sonar also can be useful in locating metallic obstructions, such as sunken vessels, pipelines, and armored cables that may hamper port or construction operations. Marine refraction techniques that measure seismic wave propagation speed can be used to obtain data on sediment and subsurface bedrock quality and depth to sound rock because propagation speed is related to material type and strength. Ground-penetrating radar (GPR) also can be used at shallow freshwater sites (generally 10 to 20 ft) for water depth and limited subbottom profiling. Table 8-3 lists several types of geophysical equipment that can be used for site characterization below water. Several types of land-operated geophysical equipment are also available to aid in the assessment of a site.

Subsurface Exploration Program Planning

There are no hard and fast rules as to how many explorations are required, where they should be located, or how deep they should penetrate. These decisions are different for each project and depend upon available information gathered during the site reconnaissance, as well as the project location and configuration, structure type, local geology and subsurface conditions, and size and cost of the project (Bowles 2009, Das 2011, DOD 2012). Geophysical methods can be used to help locate preliminary borings and to reduce the overall drilling program, should uniform conditions be encountered. In general, preliminary borings are taken along the face of proposed bulkheads, quays, and marginal wharves at a spacing not exceeding approximately 200 ft. For anchored bulkheads, another series of borings usually is required at the location of the anchor system. Additional boreholes may be drilled offshore and upland to obtain information on alignments perpendicular to the structure. This configuration may be desirable for the development of subsurface cross sections for stability analyses. The same scheme would be used for cut slopes and embankments running parallel to the shore.

Narrow, pile-supported piers usually require preliminary explorations along the proposed centerline, not exceeding a spacing of 200 ft, and isolated pile-supported structures and dolphins may require one or more borings at each location. Solid-fill piers and breakwaters may require additional lines of borings perpendicular to the structure centerline if stability is an issue.

The depth of the explorations depends on the type of anticipated foundation support. In the case of pile foundations, explorations must extend into the bearing layer or to bedrock, or to soil depths sufficient to develop adequate frictional resistance. Explorations for solid-fill structures, quays, bulkheads, and slopes must penetrate to below any possible failure planes. Where compressible soils exist, explorations should be advanced through the compressible layer or to a depth where the calculated increase in stress caused by the new construction approaches zero.

Table 8-3. Geophysical Methods

Method/Equipment	Purpose	Capabilities	Comments
Fathometer (depth sounder)	Determines bathymetry	Typical depth range between 0 and 500 ft	Usually operates in the 200-kHz range; results in little subbottom penetration
Bathymetry—depth or echo sounder	Determination of water depth along survey transects	Typical depth capability between 2 and 400 ft	Commonly uses 100- to 220-kHz frequency range. No subbottom penetration
Dual-frequency depth sounder	Determination of water depth and thickness for "fluff" and/or soft, high-water-content sediments	Typical depth capacity 2- to 400-ft depth range	200 kHz high frequency with 20 to 40 kHz low frequency
Multibeam systems	Full-coverage mapping of underwater bottom areas	Typical depth capability to 500 ft	450 to 650 kHz provides measured depth information as dense as one sounding per square foot
Seismic reflection profiler	Determination of subbottom stratigraphy	Operates in a wide range (5 to 500 ft) of water depths with penetration depths up to 500 ft depending on operating frequency	Typical operating frequencies range from 1.5 to 16 kHz. Lower frequencies provide deeper penetration with less resolution
Side-scan sonar	Locating and defining bottom surface features	Scan (sweep) range of 50 ft to a few hundred feet to each side of towed transducer	Typical operating frequency of several hundred kHz. Transducer (transmitter and receiver) towed 10% to 20% of sweep width above bottom; sound waves transmitted from either side of tower equipment
Magnetometer	Determines location, estimates size and depth of surface and buried metallic objects. Maps large magnetic anomalies associated with large-scale geologic or manufactured features	Operates in any water depth	Equipment towed near water surface or at depth; operates in any water depth. May be towed closer to mudline in search of small objects

Where high bearing pressures are a concern, the explorations should extend to a depth below the bottom of the structure equal to two times the structure's width. If deep dewatering is required, explorations should extend to at least twice the depth of the required dewatering. The total number and depth of borings required depend on the preliminary exploration information and the type of structure to be constructed. Depending on the scope and results of the preliminary drilling program, additional borings at closer spacings may be required. Test pits may supplement initial explorations where detailed shallow subsurface information is required.

Because of the high cost of mobilizing certain drilling rigs, it may be more cost-effective to adjust the exploration program as drilling progresses in order to complete the program in one phase, rather than two or more phases. Table 8-4 presents guidelines for the spacing and depth of explorations generally required for different types of structures. It must be noted that these are only guidelines and that changes to the exploration program often result from the subsurface conditions encountered.

Table 8-4. Subsurface Exploration Program Planning

Structure/Site Configuration and Depths	Exploration
Large undeveloped waterfront site	Preliminary borings 200 to 500 ft on center, advanced to competent bearing soils. Layout of final phase of borings should reflect proposed construction locations, foundation requirements, and construction procedures.
Filled sites	Exploration program should consist of borings supplemented with test pits. Proposed boring locations and depths should be based on the proposed development, and allowance should be made for slow drilling or refusal due to obstructions. Test pits should be used to define vertical and lateral extent of fill and classification of the material.
Embankments, retaining structures, piers, wharves, etc.	Preliminary borings spaced at approximately 200 ft on marginal wharves and centered along the length of the structure. Final borings at 50 to 100 ft on center, with additional borings inboard and outboard to develop cross sections for stability analyses, bedrock profiles, dredging, etc. Borings to be used for stability analyses should penetrate all possible failure surfaces.
Pile foundations	Borings should be carried into dense soils or to bedrock for end-bearing piles, or deep enough to develop adequate frictional resistance for friction piles.
Shallow foundations	For individual footing or mat foundations, borings should be completed to a depth equal to or greater than twice the width of the loaded area.

8.2 Soil Types and Characteristics

Subsurface soils found in the nearshore environment typically include sand and gravel, inorganic and organic silt, clay, peat, and fill. The soils may be relatively uniform and homogeneous, layered, or a mixture of two or more soil types, which may also include cobbles and boulders. These materials possess varied engineering properties that can affect the design, construction, and performance of structures. The engineering properties of a soil may remain constant with time, or can vary with changes in stress, pore water pressure, and physicochemical reactions (Mitchell 1976). The location, type, and characteristics of the bedrock also can play an important role in design.

The following sections briefly discuss general soil types and describe some of the more important material properties and their implications in the design and construction of nearshore structures. A brief discussion of rock properties and their engineering implications also has been included. More complete discussions of soil and rock mechanics and properties can be found in various references (Bowles 2001, 1984, Holtz et al. 2011, NAVFAC 1986a).

Soil is by nature nonhomogeneous and anisotropic, displaying markedly variable physical and mechanical properties, even when samples are obtained from the same deposit in close proximity. This variability is a result of the soil's composition, formation, and geologic and stress history, as well as physical and chemical changes it has undergone, and its current location and state of stress. Fortunately, many soils of comparable makeup display similar behavior when subjected to changes in stress from structural loads.

Natural soils transported to and deposited in the nearshore environment generally are terrigenous, consisting of gravel, sand, silt, and clay, often with the inclusion of larger cobbles and boulders (Rocker 1985). These materials are predominantly riverborne, and may be either well sorted and stratified or mixed and reworked because of currents and coastal processes. Coarser materials tend to settle out first, with finer particles carried on to estuaries, embayments, and, eventually, offshore, where they are finally deposited (Nacci 1975). Soil types, as well as the geologic and coastal history of a particular site, can profoundly affect foundation design.

The following paragraphs briefly describe general soil types commonly found in the nearshore environment, as well as their implications in the design of marine structures. Many special subsurface conditions are indigenous to specific geographic areas, but their description is beyond the scope of this book.

Sand and Gravel

Sand and gravel deposits found in the nearshore environment may be stratified by glacial meltwater flow or river processes, or may be highly mixed from subsequent reworking. These soils also are found in various gradations and may contain silt, clay, boulders, and cobbles.

Sand and gravel generally provide good bearing resistance for either shallow or deep foundations if they are relatively dense. The presence of appreciable amounts of cobbles or boulders, however, can lead to difficulties in the construction of deep foundations or in the driving of sheeting.

If relatively free of silt and clay, sand and gravel can be used as free-draining backfill for retaining walls and solid-fill structures. Piping can become a problem, however, in very clean sand where large differential hydraulic head conditions exist (see Section 8.5).

If clean, uniform, fine sand is present in a loose state below the water table, whether as a natural deposit or from hydraulic fill operations, it may be susceptible to liquefaction. Liquefaction is the sudden loss of strength that takes place in loose sands and some inorganic silts when they are quickly densified. As the granular particles move into a denser packing, water is forced out of the pore spaces. When this occurs quickly and the water cannot escape fast enough, the pore pressure increases and the effective stress decreases. Since shear strength is directly related to effective stress, the soil rapidly loses strength and becomes "quick," commonly known as "quicksand." Earthquake events (Youd and Idriss 1996) or other vibrations may lead to liquefaction that results in an almost complete loss of strength. Liquefaction can cause large settlements to the ground surface and structures, increase loads against retaining structures, and cause underground tanks and enclosed structures to literally float to the ground surface. If saturated, loose, fine sand is present, deep densification by one of several ground modification techniques (discussed in Section 8.9) may be required.

Cyclic loading of granular soils caused by wave action may cause a fatigue-type failure at shear stress levels below the ultimate static stress levels. Cyclic shear testing data can be used to determine the soil pore pressure response relative to stress levels and the number of loading cycles. This information then can be applied to stability analyses for structures subjected to multiple loadings (Frankel and Pollalis 1979). Research is ongoing on this subject, with the recent construction of many large offshore gravity-type structures (Watt 1978).

Because it is difficult to obtain undisturbed samples of clean sand and gravel, sophisticated laboratory tests normally are not performed on these materials. Values for their permeability, shearing angle, and in situ unit weights generally are inferred from grain size distribution, SPT values, or field cone results (Robertson 1986).

Inorganic Silt

Inorganic silt can exist in relatively homogeneous deposits or *varves*, interbedded with clay or fine sand as a result of the cyclic depositional environment of glacial lakes. Depending on its actual makeup and stress history, a silt deposit may behave predominantly as either a cohesive or a cohesionless material.

Deposits that resemble a sand or clay usually can be treated as such. However, particular attention must be given to the microcharacteristics of the deposit and the orientation of its planes of weakness. This is of particular importance when stability is an issue (Nacci 1969).

Uniform inorganic silt deposits can exhibit appreciable strength values even though natural water contents are near or above the liquid limit (Nacci 1969). These soils, if not heavily preconsolidated, tend to be somewhat sensitive, however, and may lose much of their strength when disturbed. For this reason, it is difficult to obtain high-quality undisturbed samples for laboratory testing. Loose silt also may be susceptible to loss of strength because of cyclic loading.

Some denser or overconsolidated silt deposits are adequate for the support of shallow foundations, provided that disturbance is minimized. A working mat of filter fabric and crushed stone often is required to avoid disturbance by construction equipment. Preloading of silt often is successful in reducing postconstruction settlements. Many silt deposits tend to be relatively free-draining, especially when appreciable amounts of sand or numerous thin sand strata are present.

Low-capacity pile foundations also can be successfully installed in silt deposits. These soils may lose much of their strength when disturbed during pile driving, but load tests show that they regain much if not all of their original strength with time.

Clay

Thick deposits of clay are typical in many nearshore environments. Strength, compressibility, sensitivity, and mineralogic makeup often vary widely from region to region, as well as with depth within a deposit. Clay can exhibit relatively high compressibility and low permeability, resulting in large settlements over long periods of time following loading; so it often is not practicable to preload a site underlaid by thick clay deposits. Many clays, however, are overconsolidated and produce small settlements for loadings below their maximum past pressure.

Lightly loaded structures may be founded on friction piles, and, in some cases, caissons can be founded directly on stiff clay. Deep foundations may be used to transfer heavy structural loads to strata below the clay deposits. If the site grade is raised, however, downdrag forces (i.e., negative skin friction) resulting from consolidation of the clay may be transferred to deep foundations (Lambe and Wolfskill 1973) (see Section 8.7).

Stability is often an issue when clay deposits are present. The stability of slopes and nearshore structures depends upon the shear strength of the soil in question. Some marine-deposited clays exhibit high sensitivity (ratio of undisturbed shear strength to remolded strength). These soils may have relatively high undisturbed strengths at high water contents but may suddenly lose most if not all of their strength during loading. This brittle type of failure is likely caused by the breakdown of bonds between the soil particles.

Uplifted marine clays that have been leached with freshwater also display this phenomenon caused by a breakdown of cementation and physicochemical bonding

(Bjerrum 1967). Clays that have not been uplifted above sea level also may exhibit these characteristics because of leaching by freshwater from artesian conditions. Sensitivities of normally consolidated marine clays typically range from 4 to 8 but may be greater than 100 for highly sensitive deposits (Nacci 1975).

Cyclic loading also can be an issue in working with clays, causing stress reversals that can increase strain and lower ultimate shear strength (Sangrey et al. 1969).

Organic Silt and Peat

Many waterfront environments are frequently overlaid by deposits of organic silt or peat. In some areas, these materials have been buried by subsequent natural deposits or filling. Most organic soils tend to be soft and highly compressible, exhibiting low shear strengths. Water contents are usually near or above the liquid limit, so that the soil approaches the liquid state with loading or disturbance (Pierce and Calabretta 1978).

The presence of organic soils can have a major effect on development because of their inherent weakness and loss of strength when loaded and their high degree of compressibility. Low shear strength can lead to the instability of slopes or extremely high pressures against earth-retaining structures. High compressibility can result in excessive settlement of shallow foundations, utilities, and bulkhead anchor systems, or it can cause sizable downdrag forces on deep foundations.

Ground modification techniques, such as deep dynamic compaction or vibratory methods, usually have little effect on organic soils (see Section 8.9). If located close to the ground surface (20 to 30 ft maximum), these soils can be removed and replaced with clean granular soils that can be treated using deep-densification techniques. If these soils are located deeper than approximately 30 ft, their removal can be problematic because of high cost, difficulty of handling, slope stability or earth support concerns, and disposal considerations. Preloading is not generally viable for organic soils with high peat content because of relatively large amounts of secondary settlement and long-term decay. Some organic silts, however, can be successfully preloaded. With the help of sand or wick drains and reinforcing fabrics, preconstruction settlements may be reduced to the extent that these portions of the site can be suitable for staging, storage, or stockpile areas, or for tanks, small buildings, or warehouses. Another alternative is the use of stone columns or rammed aggregate piers, which can aid in soil drainage while adding stability to the site. Displacement-type cast-in-place concrete columns, which, like piles, transmit surface loads to below the poor soil deposits, are another alternative (see Section 8.9).

Fill

Many waterfront sites are located on land created by filling, frequently in an uncontrolled manner. The fill may consist of a miscellaneous and random mixture of soft or loose soil containing large boulders, blasted rock, construction debris and trash, or hydraulically placed sand and silt. The amount and the depth of fill present likely reflect the original topography and required grade elevations. In many instances, the fill was placed directly over soft organic soils. Existing fill at many sites is unsuitable for supporting shallow foundations in its current state and may require ground modification or deep foundations. If the fill contains large obstructions, even the deep-foundation scheme may be a costly and difficult alternative.

Bedrock

The bedrock characteristics that are usually of concern to the marine designer include the degree of weathering and the amount of fracturing of the bedrock surface. These characteristics are important in the design of piles and sheeting that must be founded on the rock, in determining the capacity of rock anchors, or where rock excavation or dredging is required. Also, if slopes must be cut into the rock, the degree and the direction of fracturing are important in the evaluation of block stability and the need for rock bolting. The degree of weathering and fracturing can be determined from high-quality rock cores, but fracture orientation is more difficult to determine and may require directional drilling and oriented core techniques if local bedrock exposures are insufficient or unavailable for inspection.

8.3 Marine Foundations

The development of port or ship construction and repair facilities generally requires some modification of the natural shoreline in order to handle deep-draft vessels. This modification may be severe, requiring massive dredging and filling and the construction of bulkheads or marginal wharves. In contrast, the shoreline may remain relatively unchanged, with the construction of piers and dolphins for handling vessels offshore, in naturally deep water. Even in the latter case, however, some shoreline protection usually is required to prevent scour and erosion.

The design of nearshore facilities is a soil–structure interaction problem where the structure is often also the foundation. Waterfront structures are exposed to relatively severe berthing and mooring loads as well as dynamic environmental forces. These conditions can cause large lateral and uplift forces that, in addition to the gravity loads from backfill, stockpiled materials, cranes, and other handling and transfer equipment, must be resisted by the structure. Such facilities may also require deep draft capacities, resulting in long, unsupported pile lengths or high retaining walls. To further complicate the design, construction materials are subject to ongoing degradation, which decreases their structural integrity.

The remaining discussion in this chapter is devoted to design considerations for nearshore structures from a foundation engineering standpoint. Although geotechnical considerations may dictate the use of a particular structural type, cost and other factors may favor other alternatives. General design guidance for marine foundations can be found in BSI (2012), DOD (2012), API (2011), and NAVFAC (1986b).

Structural Type

There are many types and configurations of waterfront structures, each of which presents different geotechnical design issues. These structures include slopes, piers, wharves, bulkheads, quay walls, dolphins, and dry docks, and may be constructed of soil, stone, masonry, concrete, timber, or steel. Fig. 7-1 depicts some of the many types of waterfront structures, and Chapter 10 presents a discussion of several types of dry docks.

Slopes differ from other structures in that soil and rock generally provide the building material as well as the foundation. Slopes can be either cut into existing soil or constructed above in situ soil using other fill materials. A major concern with all slopes is stability, a situation that can be aggravated by erosion, scour, or surface sloughing. Slope protection, therefore, becomes a major consideration in construction along the waterfront.

Retaining structures provide a vertical or almost vertical wall that retains backfill. They may be constructed to allow dredging for berthing and mooring of vessels, to allow for land reclamation, or to provide support in front of deteriorated existing earth retention structures. Retaining structures include flexible bulkheads, mass masonry or concrete gravity walls, steel cofferdam bulkheads, and concrete caissons. The designer of retaining structures generally is concerned with issues of bearing capacity, sliding, overturning, deflection, and stability, with additional concerns such as bending stress, toe resistance, anchor design, and interlock tension for sheet pile structures. Scour and erosion often lead to the failure of bulkheads and retaining structures and must be considered in the design.

Solid-fill piers and dolphins are similar in design to many of the solid-fill retaining structures and share many of the same structural features. The major geotechnical difference is that, in most cases, the piers and dolphins are not used to retain backfill behind the structure itself. Alternatively, open pile-supported construction can be used for piers, wharves, or dolphins. Because of their relatively light weight, these types of structures generally rely on batter piles to resist lateral forces. Major geotechnical concerns for pile design are compression, uplift, and lateral loads.

Foundations for dry docks, such as marine railways, vertical lifts, and floating dry docks (see Chapter 10), can take on many different forms. Marine railways may be supported on either shallow or deep foundations, or a combination of the two; winches for vertical lift systems generally are supported by open, pile-supported piers; and floating dry docks may be moored to solid or open-type piers, wharves, or dolphins. The walls of graving dry docks generally take on the form of one of the above-referenced retaining structures. Buoyancy and uplift pressures are of great concern with this type of structure. Shallow, narrow docks can be designed to resist these forces with the use of dead weight and structural floors. However, most major basin docks require either tension piles or grouted anchors to reduce spans and counteract buoyancy forces unless a pumped underdrain system is incorporated.

Construction Considerations

Construction near or over the water tends to be more difficult and costly than upland construction, and the logistics of handling equipment and materials becomes an important consideration. Work that can be performed from land usually is more cost-effective than similar work performed using barge-mounted equipment. Environmental conditions such as tides, currents, and wave action must be constantly dealt with and may require the construction of temporary dikes or cofferdams. Additionally, ongoing shipping and handling operations can further hamper construction.

Construction operations and equipment used in waterfront work often differ from those used on land. Pile driving operations may require the use of free instead of fixed leads, and elaborate driving templates often are needed to ensure alignment and proper mating to other structural members. Backfilling operations can be slow, requiring the use of clamshell or conveyor equipment to place materials. Compaction of the backfill may require the use of one or more of the ground modification techniques discussed in Section 8.9. Dewatering for deep structures also can be difficult and expensive. Often, the construction of extensive cofferdams is the only effective way to perform the work.

The design of waterfront structures must take these factors into account in order to obtain the most efficient and economical construction. The geotechnical portion of many marine projects is large and requires careful planning. Facilities may be designed so that subsequent portions of the work can be constructed by using completed portions for staging of equipment and materials. This approach may dictate the type of construction or the sequence of filling and dredging operations.

8.4 Slopes and Slope Protection

An important consideration in slope design is stability, which depends on soil and rock characteristics and controls the steepness and configuration of the slope. Generally calculated in terms of the factor of safety against failure, stability can change with time. Changes in the factor of safety can occur as a result of the construction sequence; the pore pressure response during or after construction; fluctuations in the groundwater table; surcharge loadings; changes in slope configuration caused by scour, erosion, and dredging; or earthquake events.

Problems with scour and erosion caused by current and wave action are common along the shorefront. For this reason, the exposed faces of slopes must be adequately protected. A range of slope-protection examples are shown in Figs. 8-3 and 8-4. Additionally, many waterfront land-reclamation projects are constructed over soft soils or use weak or hydraulically placed soils as backfill (Enkeboll and Smoots 1980). These situations can lead to severe stability problems and must be carefully engineered to avoid failures.

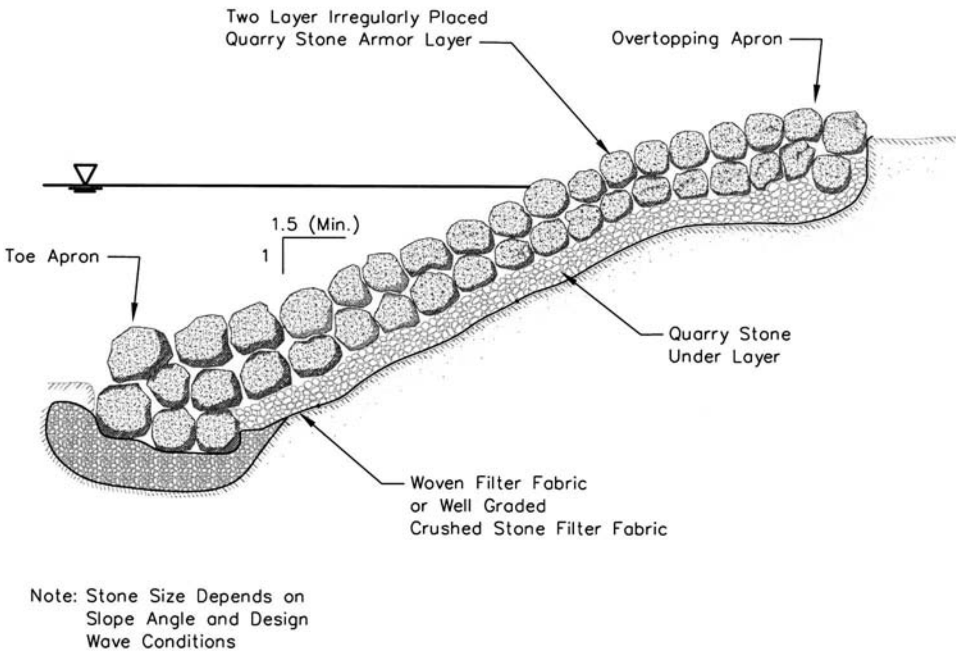


Fig. 8-3. Typical riprap slope protection

Analysis of Slopes

There are many different modes of failure and associated methods of analysis for slopes constructed of earth or rock (Huang 1983, Duncan 2005). In general, each method compares the forces tending to cause failure (driving forces) to the forces tending to resist failure (resisting forces) in order to determine a factor of safety against slope failure or sliding. All of these methods can be calculated either by hand or by utilizing one of the numerous computer programs that have been developed to assist in the process. Details of the calculations can be found in Huang (1983). All slope analyses should also take into account additional driving forces due to equipment, product, stock-piled materials, and earthquake events.

Natural erosion or scour at the toe of a slope or dredging can affect stability. Seepage pressures are also a concern for waterfront slopes, especially in areas subject to coastal flooding. Such flooding can substantially raise the groundwater level within the slope, producing increased driving forces and seepage pressures if the flood waters subside quickly.

Minimum recommended factors of safety (FS) range from 1.3 to 1.5, although lower values are acceptable under certain conditions (Huang 1983, Duncan and Wright 2005). As discussed below, the factor of safety of a slope can change with time because of the pore pressure response of the soil, which must be accounted for in the analysis. Recommended factors of safety assume that the most critical failure surface has been analyzed, that soil strength parameters are adequately known, and that

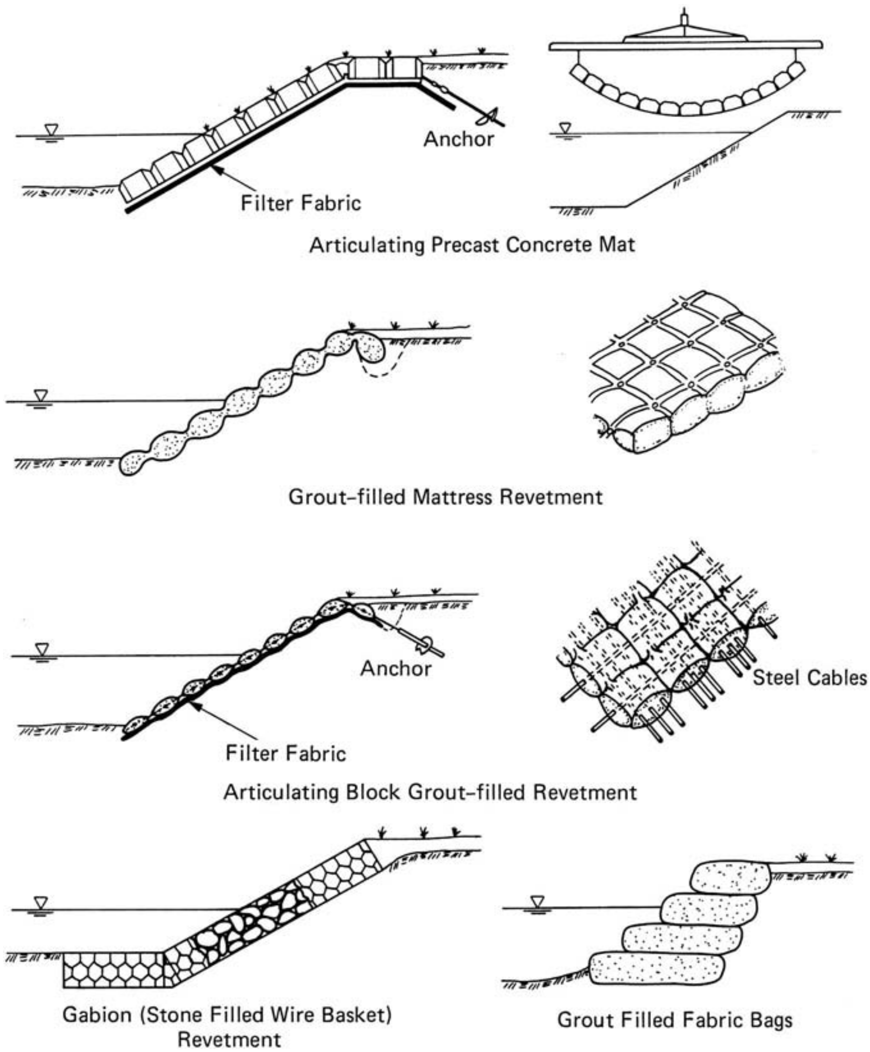


Fig. 8-4. Typical slope-protection systems for areas of low wave and current energy

good control has been maintained during construction (Huang 1983, Duncan and Wright 2005). Where there is a high potential for loss of life or property, more conservative factors of safety may be warranted.

Cuts and Fills

Although the final configuration of two slopes may be identical, their factors of safety against sliding at any point in time may be quite different, depending on construction methods and the soil pore-pressure response. The major contribution to the forces resisting failure comes from the shear strength of the soil along the failure surface. Any change of the pore pressure within the soil results in a change of effective stress, which, in turn, affects shear strength and overall slope stability.

Highly permeable granular soils are not generally affected by pore pressure because dissipation of the pressure takes place very quickly compared to the construction time. Pore pressure dissipation in cohesive soils is slow, however, and can greatly affect slope stability. When filled embankments are constructed over normally consolidated to slightly overconsolidated cohesive soils, pore pressures increase, resulting in a decrease in the shear strength of the soil, as well as a reduction in the stability of the slope. Pore pressures dissipate as the soil consolidates with time, however, and the factor of safety against sliding increases correspondingly. When cuts are made into or above cohesive soils, pore pressures become negative, increasing the short-term factor of safety of the slope. As the negative pore pressure dissipates, the soil expands, the water content increases, and the shear strength and slope stability decrease. Depending on how the slope is constructed, either the short-term (end-of-construction) or the long-term factor of safety determines the critical factor of safety against failure.

When calculated factors of safety are low, several steps can be taken to decrease the possibility of failure. The rate of fill placement can be slowed to allow underlying soils to consolidate and gain strength under the new loads. Pore pressures also can be monitored to determine when factors of safety are adequate for filling operations to continue. The rate of consolidation of underlying soils also can be increased by the use of sand or wick drains (see Section 8.9). Where the long-term factor of safety is too low, preloading of the underlying soils can be used to increase shear strengths before the construction of cut slopes. Slopes also can be benched, counterweighted, or decreased in steepness to reduce the sliding potential.

The strength of loose fills and underlying soils can be increased in place by using one of the ground modification techniques discussed in Section 8.9. In addition to the densifying and strengthening of in situ soils, shearing resistance along the failure plane may be increased by installing piles, stone columns, rammed aggregate piers, or displacement-type concrete columns through the anticipated slip surface (Dobson 1986). Also, steeper and more stable embankments may be constructed by placing reinforcing fabrics or grids within the fill (Koerner 1997).

Slope Protection

Numerous methods and materials are used to protect the exposed faces of waterfront slopes from scour and erosion caused by wave action, currents, and propeller wash. A common method is to build up graded layers of stone with a protective layer of riprap on the slope face (PIANC 1997) (Fig. 8-3). The individual stone size and thickness of the protective layers depend on the forces to which they are subjected. Exposed slopes subjected to heavy wave action also may be protected with manufactured armor units, which are carefully placed on the slope face (see Fig. 8-40). These units are made of concrete and tend to be relatively expensive to make and place, but they may provide the necessary protection in areas where large rock for riprap is not available or economical.

Several other proprietary systems of slope protection are available for use in areas where wave and current forces are limited. Fig. 8-4 shows a number of these systems.

One slope-protection system uses *gabions*, which consist of wire baskets laced together and tightly filled with stone. The baskets can be galvanized or coated with polyvinyl chloride for marine use and are available in many shapes that can be connected to form surface mats or stacked to form retaining structures or revetments. Use of thin-gauge wire baskets in a saltwater environment may be questionable, however, even with the protective coating.

Concrete-block revetments can be used in low-energy areas; the blocks are laid on the slope to provide continuous face protection (USACE n.d.). Grout-filled fabric mats are another form of slope protection that can be used where there is limited wave action. The fabric form is laid on a prepared slope, anchored, and filled with pumped concrete. The segmented mats have a quilted appearance when completed and are usually designed to allow water to pass through the seams (Welsh and Koerner 1979).

Slope-protection design must include a filter to stop migration of fine soils from below. Graded stone and gravel filter layers generally are used with the large stone riprap or armor unit type of protection. In recent years, geosynthetic filter fabrics have been used extensively in coastal engineering (PIANC 1992). They are routinely used with most of the proprietary systems to reduce labor and materials costs while providing durable and effective filtration.

The portion of a slope below low water, as well as the bottom adjacent to the toe, must be protected from scour caused by wave action, currents, and propeller wash (Herbich et al. 1983) (see Sections 3.4 and 4.6). This protection has generally been accomplished by using a layer of stone, installed to a sufficient depth below the mudline, with the material sized such that an adequate factor of safety with respect to threshold eroding velocities is maintained (Gaythwaite 1981). A layer of high-strength geotextile usually is placed between the stone bed and the existing bottom soils to avoid washout of fine materials. Concrete-filled geotextile "mattresses" also have been used below water for scour protection (Welsh and Koerner 1979).

8.5 Bulkheads and Retaining Structures

Retaining structures are used extensively in the development of waterfront facilities. These structures are used to stabilize the shorefront, allow dredging, and provide for land reclamation. Although these goals can be accomplished by creating slopes or embankments, retaining structures provide a vertical face for berthing and mooring of vessels without wasting valuable land area. Retaining walls generally are more costly than slopes, but they may be more cost-effective than open, pile-supported structures, depending on the site's physical and subsurface conditions. Even if a pile-supported wharf is used, some sort of retaining structure is generally used in conjunction with it.

General Discussion

Retaining structures used along the shorefront include cantilevered and anchored flexible bulkheads, solid-fill steel-sheet-pile structures, concrete caissons, and mass gravity-type concrete or masonry quay walls (see Fig. 7-1). Each type of structure has advantages, disadvantages, and particular design considerations that should be discussed early on with the design team to assess possible implications, risks, and structural performance. As discussed in the following sections, some general design considerations are common to retaining-type structures. Most retaining structures require stable subsurface conditions to provide an adequate foundation. When backfilled, these structures tend to be quite heavy, thereby greatly increasing the stresses in subsurface soils. The increased loads can cause bearing failures, instability, or intolerable settlements. In addition, these structures can severely limit future deeper dredging, unless they are specifically designed for it.

Retaining structures also must resist lateral earth pressure loads. These loads increase with the square of the height of the wall, thus requiring heavy sheeting and anchoring or excessively large gravity structures for high walls. Wall heights of approximately 50 ft for anchored sheet pile bulkheads, 60 ft for concrete caissons, and 80 ft for sheet-pile, cofferdam-type construction are practical limits. Mass gravity concrete or masonry walls, rarely used today because of their relatively high cost, are generally less than 40 ft high.

Lagging water levels behind a retaining structure increase lateral forces significantly (Fig. 8-5). In addition, differential piezometric heads on either side of the wall cause water to flow under the structure. If the situation becomes severe enough, piping, leading to a "quick" condition, can take place. This piping causes the soil to lose its shear strength and passive resistance and can lead to a failure (Cedergren 1989). To avoid this situation, retaining structures should provide for free drainage of water from behind the wall. This drainage is usually accomplished by using free-draining backfill above extreme low water and providing adequate weep holes as close as practicable to that level. Alternatively, a conservative water lag can be incorporated into the design. In any case, where there is any question of drainage, structures should be designed for a minimum water lag of 25% to 50% of the tidal variation.

NAVFAC (1981) recommends that a tidal lag of 50% of the normal tidal variation be used for impermeable backfill, with a coefficient of permeability (K) less than or equal to 10^{-3} ft/min or for walls of low permeability, such as sheet piling, with backfills that have K values between 10^{-3} and 1.0 ft/min. This reference also provides a graph for determining the tidal lag for soils with various permeabilities when a permeable wall, such as masonry block or timber crib, is used. It should also be noted that even if the appropriate tidal lag is used, much of the lateral pressure above the groundwater level should be calculated using wet rather than dry soil unit weights.

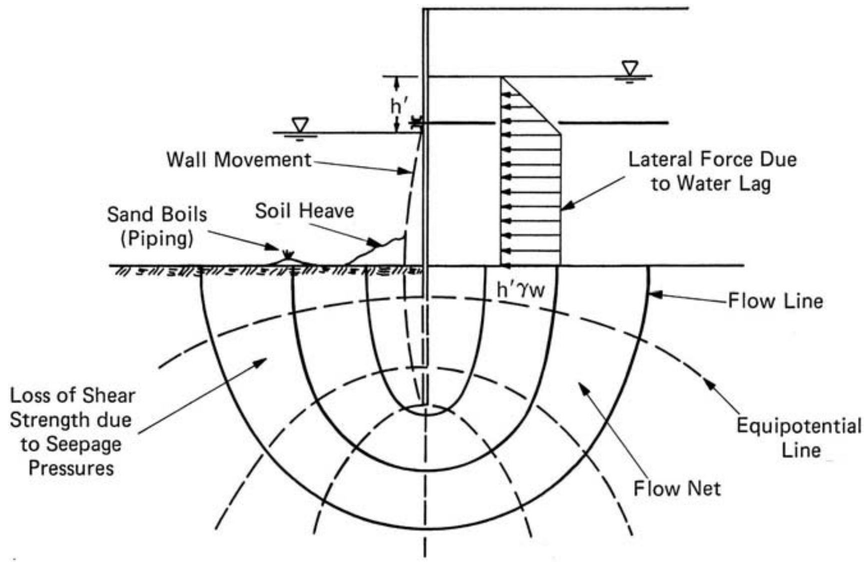


Fig. 8-5. Effect of water lag on retaining structure

These procedures or recommendations do not account for inundation of the backfill from overtopping. Where overtopping from wave action, storm surge, or flooding is possible, the structure should either provide positive surface drainage to avoid saturation of the backfill or should be designed to handle a full hydrostatic load to the top of the wall. In such situations of temporary loading, lower factors of safety, overstressing, or reduced load factors can be allowed for many of the structural members. It is imperative, however, that adequate factors of safety be maintained for critical elements of the design, such as anchor systems, resistance to piping, and overall stability.

Backfilling of the structure must be considered during design. Not only must a free-draining, granular backfill material be used, but also the method of placement and densification must be determined. Organic silt and soft sediments should be removed from immediately behind a retaining structure to avoid large lateral stresses. If existing bottom contours behind the structure are not steep, soft sediments may be displaced by progressively placing a granular fill dike immediately behind the structure. Placing fill from the landward side can aggravate the situation by causing a mud wave against the structure.

Hydraulic filling can result in high lateral loads on retaining structures during construction and generally controls the design. Hydraulically placed sand tends to settle into a loose state and may be susceptible to liquefaction during earthquake events or from other vibrations. To ameliorate the situation, densification can be performed using one of the ground-modification techniques discussed in Section 8.9.

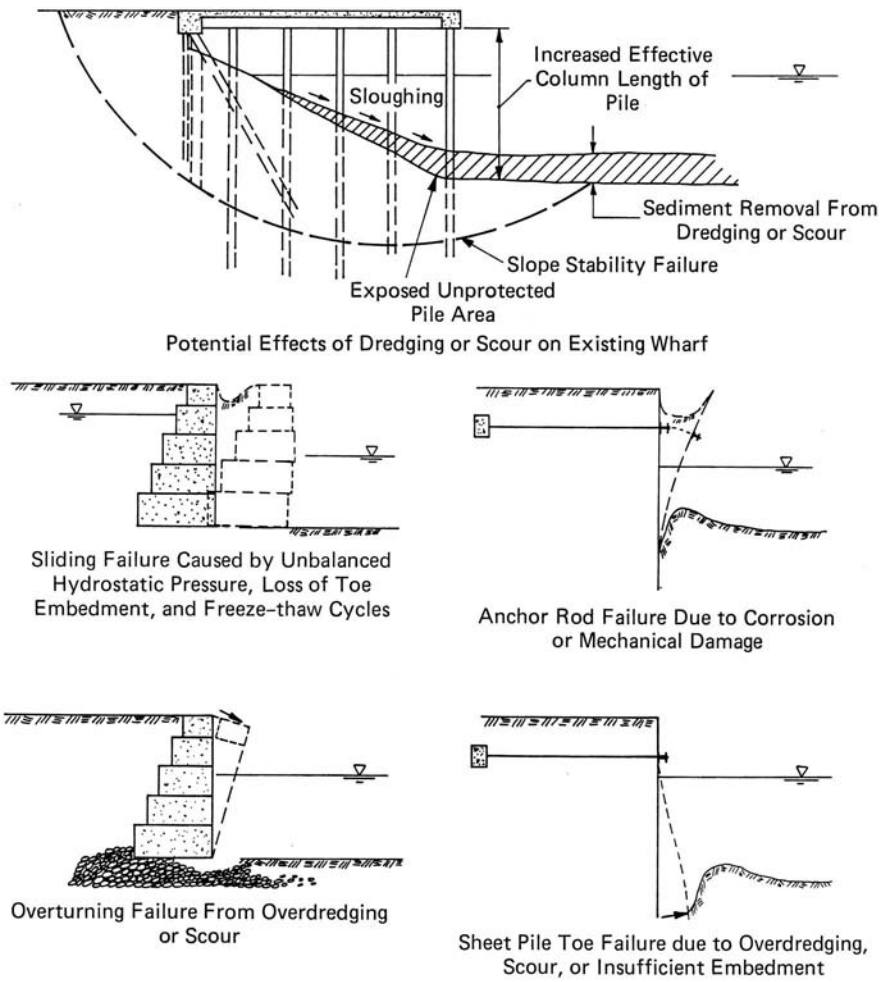


Fig. 8-6. General modes of failure of waterfront retaining structures

In general, waterfront walls fail more often because of scour or undermining at the toe or from buildup of hydrostatic pressure behind the wall than from direct environmental, mooring or berthing, or surcharge loads. Cantilevered and anchored bulkheads also can suffer serious damage or failure caused by scour, overdredging, or shallow embedment at the toe. It is important to provide protection from scour, as discussed in Section 8.4, or to design the structure for such conditions. Corroded or undersized tie-rods and connections also lead to many bulkhead failures. The design of all retaining structures should take into account lateral loads caused by an earthquake event. Fig. 8-6 depicts general modes of failure for several types of retaining structures.

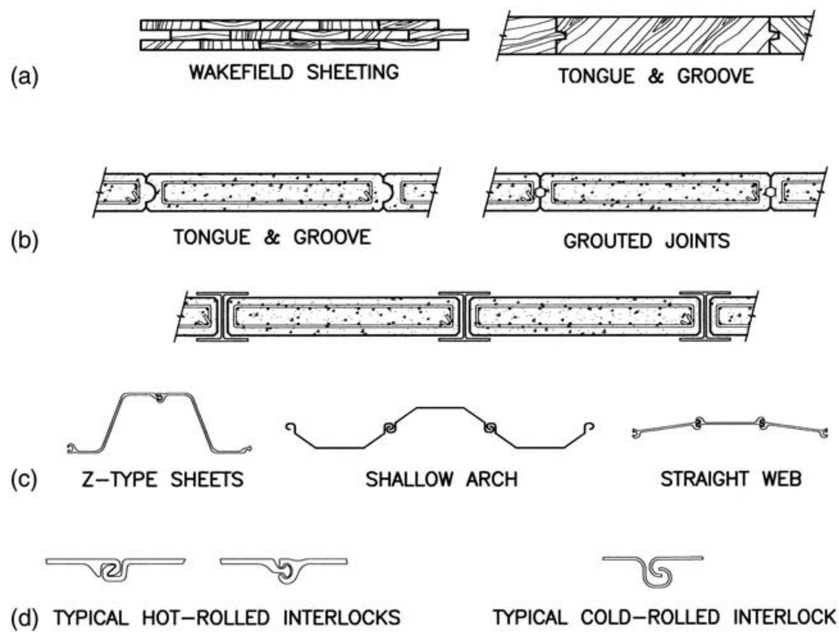


Fig. 8-7. Typical sheet pile sections: (a) timber sheeting, (b) concrete sheeting, and (c) steel sheeting

Source: Adapted from Teng (1962)

Flexible Bulkheads

Flexible bulkheads can be designed with steel, concrete, timber, plastic, or composite materials (see Fig. 8-7). The majority of bulkheads are designed using interlocking steel sheeting because of its high strength, numerous available shapes, high bending resistance, and ease of construction. Z-shaped sheets are the most common and are fabricated so as to allow for a wide range of section moduli or bending resistances. Steel sheeting has a relatively thin profile and can be easily vibrated into most soils. Steel is also durable enough to handle relatively hard driving conditions. Most steel sheets are made using a hot-rolled process that allows for the fabrication of tightly fitting interlocking joints. Tight interlocks are important to reduce the possibility of loss of backfill soil or to keep construction cofferdams dry. Cold-rolled sheets typically have loose-fitting interlocks and should be used with caution, especially in areas with tidal fluctuations or when seepage is an issue.

Prestressed concrete sheeting has been used for bulkhead construction but is less common because of installation difficulties and its relatively high cost. Precast concrete panels are sometimes used as lagging installed between driven steel H-piles.

Timber and vinyl sheeting has been used for low-capacity bulkheads. Heights are limited because of low material strengths and high deflections caused by low modulus of elasticity. In recent years, there has been significant research on the use of composite plastics in the marine environment. One of the products developed is fiber-reinforced polymer (FRP), which is being used to fabricate piles and sheeting (Lampo et al. 1998a, b). The advantage of using FRP structural members is primarily long service life compared to wood and steel, with little or no required maintenance. The current disadvantages are high cost, moderate strength, and low elastic properties as compared to steel. Longitudinal allowable stresses are in the range of 25,000 psi, and the modulus of elasticity is in the range of 4×10^6 psi. These properties limit the wall heights that may be supported by FRP sheet piles; however, with the range in section properties available, FRP sheet piles are applicable for anchored wall heights up to about 15 ft with one level of anchors.

Flexible bulkheads can be constructed in either cantilevered or anchored fashion (Fig. 7-1). Cantilevered construction usually is limited to wall heights of 10 to 15 ft because of excessive bending moments in the sheeting and deflection at the top of the wall. These structures depend solely on passive earth pressures for support and must be driven relatively deep to attain adequate support.

Cantilevered bulkheads are severely affected by scour and erosion, so they normally are used only for low walls or as end-return walls in shallow water for other major structures. The development of piling systems with large bending resistance, however, has made the use of much larger cantilevered structures possible where soil conditions are favorable and scour is not a problem. These systems include sheet pile "Combi" or king pile walls. Two of these systems include H-Z-type steel sheet piling and large-diameter prestressed concrete cylinder piles (Figs. 8-8 and 8-9).

Fig. 8-8. Installation of H-Z steel sheeting using double king piles

Source: Photo courtesy of GZA GeoEnvironmental, Inc.

Fig. 8-9. Prestressed concrete cylinder piles used as a cantilevered retaining wall, City of Newport News, Virginia, Seafood Industrial Park

Source: Photo courtesy of The Maguire Group

One common type of "Combi wall," H-Z-type steel sheet piling, has opened up a whole new realm of possibilities for bulkhead construction and rehabilitation (Carchedi and Porter 1983). The name H-Z actually describes the type of pile system, in that it is composed of a series of specially designed H-piles integrated with intermediate pairs of Z-sheets. The H-piles are sized to resist anticipated bending stresses, and the Z-sheets act as a diaphragm to transmit load to the H-piles. Because the Z-sheets are not used to develop appreciable resistance to bending, they need only be driven to a safe scour depth below the mudline.

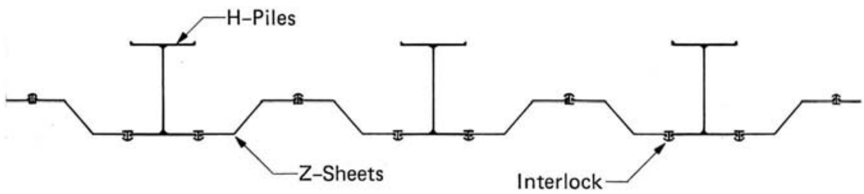


Fig. 8-10. H-Z steel sheet piling

The H-Z system is set up so that the section modulus of the wall can easily be changed by selecting from a number of different H-pile sizes. Numerous combinations of H-piles and Z-sheets fit together with a common interlock, resulting in a wide range of available section moduli (Fig. 8-10).

Anchored bulkheads derive their support from a combination of passive earth support and anchoring and are somewhat less susceptible to damage caused by scour than are cantilevered bulkheads. The wall height usually is limited by anchoring requirements rather than bending of the piling because of the availability of sheeting systems with large section moduli (Carchedi and Porter 1983, Nolan 1976). The use of multilevel tiebacks to decrease the size of sheeting and individual anchor components is also possible. This use is usually impractical in waterfront structures, however, because the lower anchor generally would have to be located well below low water to have any appreciable effect.

ASCE (1996) and Lindahl (1987) present design considerations for several types of bulkheads. Analytical methods used for the design of anchored bulkheads range from the liberal Danish rules and free-earth-support methods to the more conservative fixed-earth method. Tschebotarioff (1979) proposed a simple equivalent-beam method that assumes a hinge at the dredge line and sets the driving depth of the sheeting at 0.43 times the exposed height of the wall. This method tends to give reasonable values for the required section modulus, depth of penetration, and anchor force, which generally fall somewhere between the more conservative and liberal methods.

As piling is driven deeper, rotation of the sheeting and the required section modulus decrease. In certain cases, simply driving longer sheets may result in the use of lighter sections and overall cost savings. No matter which method of analysis is used, there is a trade-off between the anchor force and the required sheeting section modulus, depending on where the anchor is located. Generally, as the level of the anchor is lowered, the anchor force increases and the required sheeting section modulus decreases. Anchors, therefore, are normally placed as close to low water as possible in order to decrease the required sheeting section modulus, which, in turn, increases the anchor force. In most marine structures, however, the distance between the mudline and low water is much greater than the distance between low water and the top of the wall. This distance usually makes it more cost-effective to decrease the sheeting size by lowering the anchor as far as possible. Because the

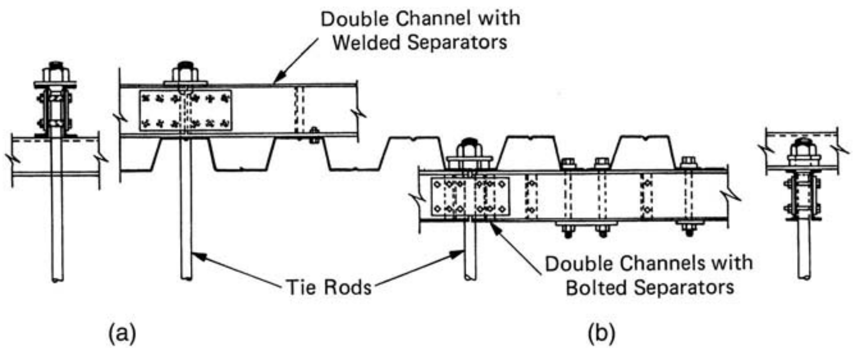


Fig. 8-11. Typical wale details for anchored sheet pile bulkhead: (a) wale located outside of wall; (b) wale located inside of wall

highest moments in the sheeting occur over a relatively short distance, the section modulus can be increased over this distance by welding plates to the flanges of the steel sheeting. Because of the labor costs involved, however, this solution may or may not prove economical.

Anchoring of a bulkhead generally is accomplished by using anchor rods that span between the wall and the anchor system. Anchor rods are spaced some distance apart, generally 5 to 12 ft, and are attached to the wall through a wale. A typical wale consists of a pair of channels spaced back to back, to which the anchor is connected. Two possible positions for the wale are on the face of the wall and behind the wall (Fig. 8-11). From a mechanical standpoint, the exterior wale is the best alternative because the piling bears directly against it, and the whole system is available for periodic inspection. From a practical point of view, however, most wales are placed behind the wall. This arrangement avoids conflict with vessels and fendering systems while affording some protection from corrosion and mechanical damage. Care must be taken in the design of this type of installation because the wall is in essence supported by a series of bolts rather than the wale system itself.

Many types of systems can be used to anchor a wall, as shown in Fig. 8-12. Generally, the most economical system uses a *deadman*, which uses passive earth resistance to provide anchorage. The deadman can consist of concrete blocks, a continuous concrete wall, or driven sheet piling with another wale system. Use of this type of system may be difficult when the proposed bulkhead is located a good distance from the existing shorefront. Other anchor alternatives consist of battered pile anchors for the tie-rods, batter piles (either tension or compression) attached directly to the wale, or drilled and grouted anchors.

The location of the anchor system is also of particular importance, as shown in Fig. 8-13. An anchor placed within the active wedge of soil behind the wall provides no resistance. A deadman system must be placed behind line *OB* in order to provide

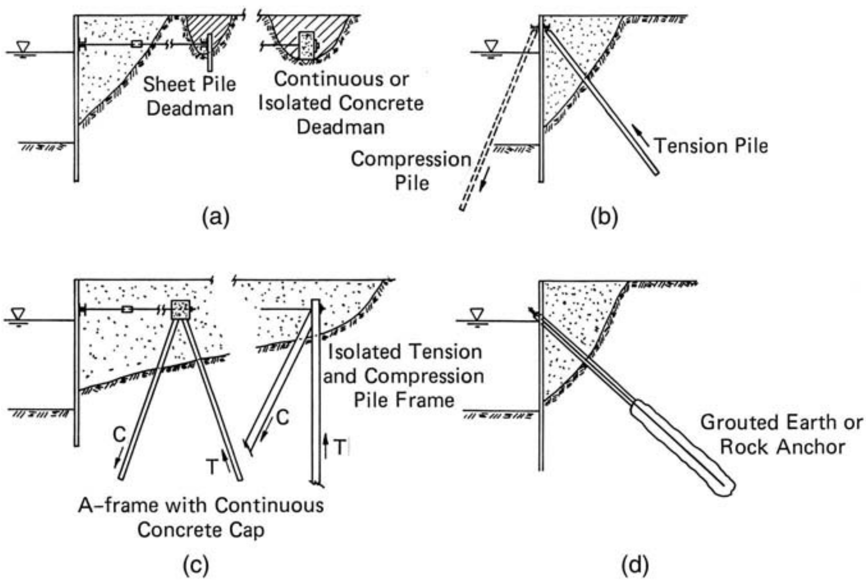


Fig. 8-12. Typical bulkhead anchor system: (a) tie-rods and deadman; (b) H-pile tension or compression anchor; (c) tie-rods and pile anchors; and (d) drilled grouted anchor

Source: Adapted from Lindahl (1987)

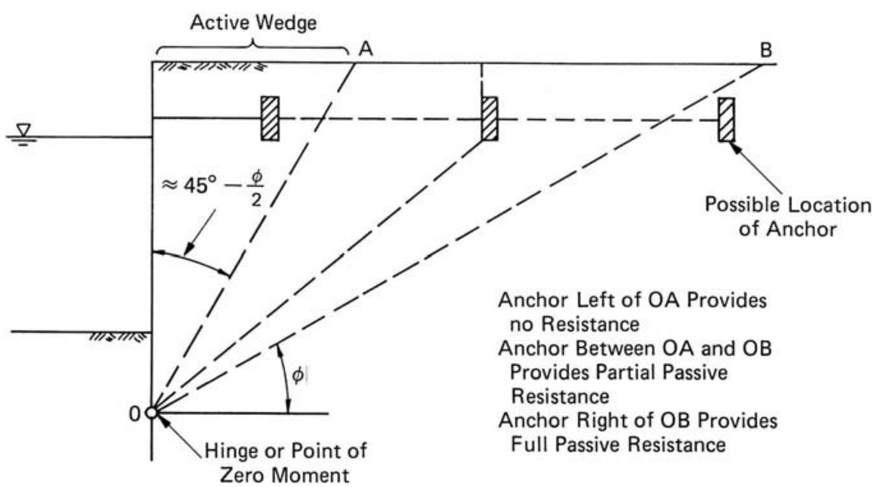


Fig. 8-13. Effect of location of anchor system

Source: NAVFAC (1986b)

full resistance without placing additional load on the wall (Lindahl 1987). Pile-supported systems may be placed between lines *OA* and *OB* because resistance is derived from compression and tension in the piles rather than from passive earth pressure. When tension pile anchors and grouted anchors are used, only the zone outside the active wedge can provide resistance. Several other possible modes of failure and design considerations must be addressed when grouted anchors are used, as discussed in Post Tensioning Institute (2014).

All anchor systems should be designed for a minimum factor of safety of 2.0. This factor provides redundancy in the system so that if one anchor fails, progressive failure of the whole structure does not take place.

Recent advances in soil–structure interaction (SSI) models using finite element methods have provided an analysis option to develop a better understanding and estimate of stress–strain with the soil retaining structure system and to predict deflection of the structure and soil mass. It is readily applicable to flexible bulkhead applications. This method readily accounts for the elastic-plastic nature of soils, complex load geometry, and loading types. The software Plaxis is a popular software code specifically developed with SSI modeling capabilities. More detailed information using SSI models can be found in the Plaxis user manual (Brinkgreve et al. 2013).

Solid-Fill Gravity Walls

Two major types of solid-fill gravity structures are in common use as waterfront retaining walls: sheet pile cofferdam-type structures and concrete caissons (Fig. 7-1). Both systems resist overturning and sliding by virtue of their weight and dimensions, and they must be analyzed for bearing capacity failure and stability when founded on soil. These structures provide a particular advantage over anchored bulkheads in that they can be designed to resist large lateral loading and are suited for use where shallow rock exists.

Sheet pile cofferdams are constructed with straight web or shallow arch-type sheeting and, as shown in Figs. 7-1, 8-14, and 8-15, are generally driven in the form of interconnecting circles or diaphragms. The circular-type construction has the advantage that each individual circular cell can act as an independent structure, thus greatly facilitating construction. Diaphragm bulkheads must be constructed in stages because excessive filling within an individual cell causes collapse of the common diaphragm. Proper alignment during driving is critical to achieve closure of the cells and connection with adjacent cells. Driving templates are used to ensure cell configuration. Soft organic sediments must be removed from within the cells before the filling to avoid excessively high lateral stresses.

Circular cells act in hoop tension, which is dependent on cell radius and lateral stress. Cells are therefore limited in size and height by the allowable interlock tension in the piling. Diaphragm cells, however, can reduce hoop tension by using a smaller arc radius. Thus, this configuration allows greater width of the structure, which translates into greater lateral resistance and allowable height.

Cellular structures must be analyzed for several modes of failure not common to other gravity-type structures, such as slippage between the sheeting and cell fill, shear failure within the fill, and interlock tension. Design details and results of laboratory

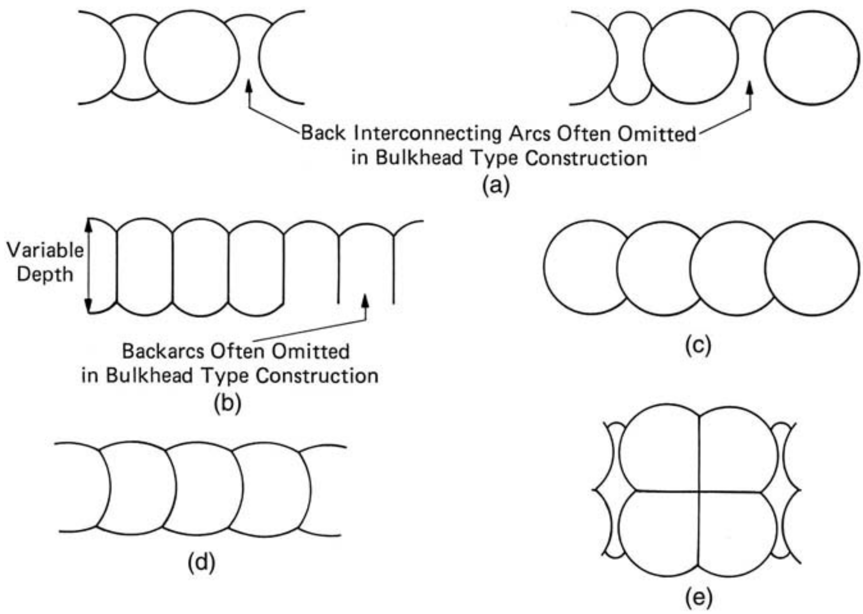


Fig. 8-14. Steel-sheet-pile cellular structures used for cofferdams, bulkheads, or dolphins: (a) circular cells with interconnecting arcs; (b) diaphragm cells; (c) modified diaphragm cells; (d) modified diaphragm cells; and (e) cloverleaf cells

Fig. 8-15. Circular steel-sheet-pile cofferdam bulkhead, showing construction sequence, New Bedford, Massachusetts, Marine Commerce Terminal

Source: Photo courtesy of Apex Companies, LLC

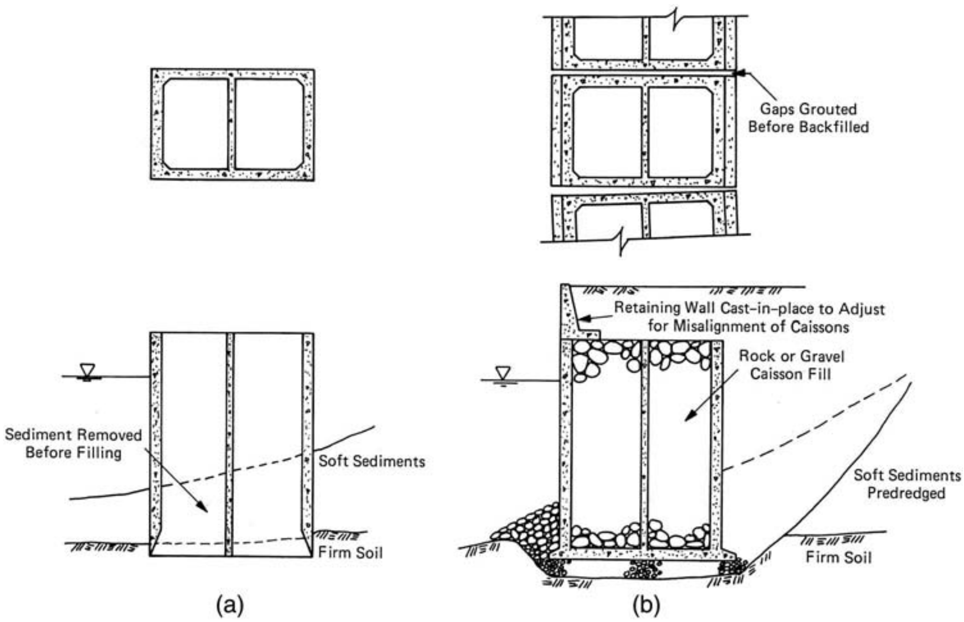


Fig. 8-16. Concrete caissons: (a) open-bottom and (b) closed-bottom

and field testing of cellular structures are provided in Lindahl (1987) and Schroeder and Maitland (1979).

Concrete caissons are prefabricated structures built on land or in dry dock and transported for placement at the site (Fig. 8-16). Caissons can be either open- or closed-bottom. Open-bottom caissons with cutting edges are lowered into place, usually in sections, and are sunk into the sediments to firm bearing. Sinking may be facilitated by predredging or jetting. Closed-bottom caissons are usually floated to the site, then ballasted to a prepared bed of scour-resistant crushed stone or rock. Both types of structures then are filled with rock or granular fill. Because of difficulties in attaining precise alignment during placement, filters or grout must be placed between adjacent sections to avoid washout of backfill. The top portion of these structures is generally cast in place to adjust further for misalignment.

Quay Walls

Mass masonry or concrete quay walls also are designed as gravity structures. This type of wall, although long lasting, is costly to construct and thus is rarely used today. These designs may be competitive for smaller walls, however, if precast concrete sections are used or if construction can take place in dry conditions. Precast cantilevered or L-type walls also may have merit for smaller wall applications. At locations where bearing capacity or settlement is a problem, quay walls may be supported on pile foundations.

8.6 Solid-Fill Structures

Solid-fill structures are often used to construct piers or breasting and mooring dolphins. A solid-fill pier can be a massive structure providing several acres of land reclamation for upland activities, or it can be a relatively narrow structure for handling vessels and product. The mass-fill-type structure incorporates one or more of the retaining systems discussed in the previous section to form the exterior of the pier. Typical solid-fill pier configurations are shown in Fig. 8-17. Isolated dolphins often are required for the breasting and mooring of vessels. Circular steel sheet pile cells commonly are used for this type of structure (Fig. 8-18).

Design Considerations

The design concepts and considerations applicable to solid-fill piers and dolphins are similar and in many cases identical to those outlined in Section 8.5. Mass-fill structures of sufficient width simply are designed as normal backfilled retaining structures. The design of narrow piers, however, may be controlled by exterior loadings rather than internal forces. These structures, generally constructed as a series of interconnecting circular steel-sheet-pile cells, concrete caissons, or

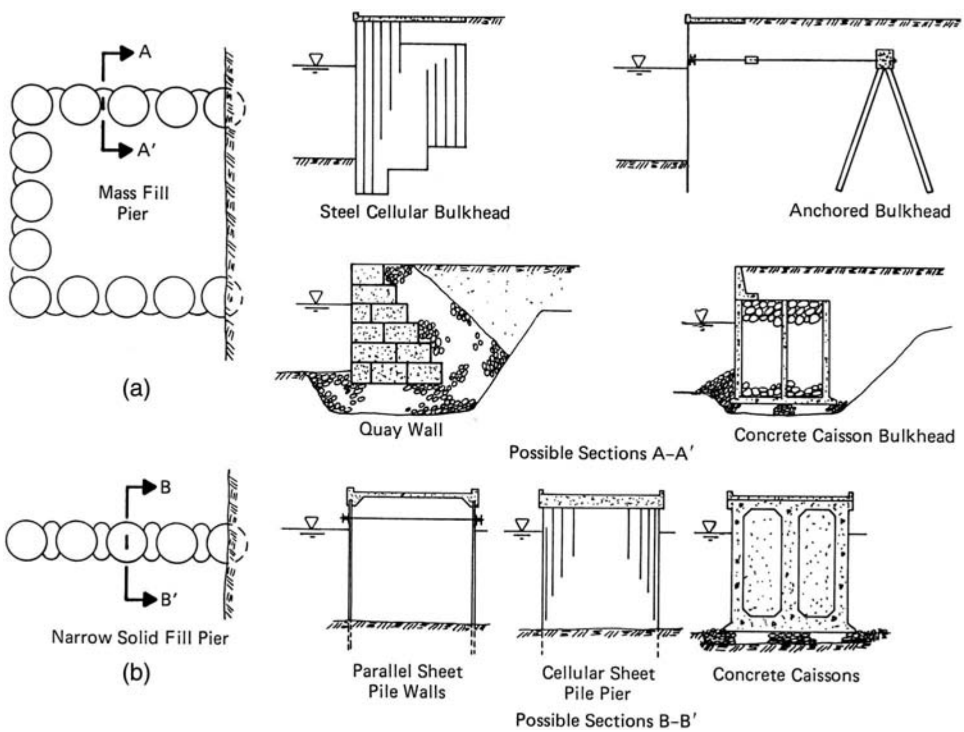


Fig. 8-17. Solid-fill pier configurations: (a) mass-fill pier; (b) narrow solid-fill pier

Fig. 8-18. Solid-fill steel-sheet-pile dolphin

Source: Photo courtesy of Childs Engineering Corporation

anchored parallel sheet pile walls, are sized by the overturning resistance required for berthing, mooring, and wave forces. Because of the narrow width and heavy gravity load, bearing capacity or stability may become the controlling issue.

Solid-fill dolphins are commonly constructed as isolated circular sheet-pile cells. Where the water is deep and lateral loadings are high, a cloverleaf-type configuration may be used to reduce hoop tension while increasing stability (Fig. 8-14). Dolphins used for breasting and turning of vessels are subjected to large torsional as well as lateral forces that can result in racking and twisting of the cell and rupture of the interlocks. An exact analysis of interlock stress for these conditions is difficult because of the nature of the structure. These effects, however, can be reduced by stiffening the top of the cell with a deep cast-in-place concrete cap, by welding interlocks together, or by outfitting the dolphin with a low-friction fendering system (see Chapter 5). Isolated cells also are used to support loading and cargo-handling facilities, catwalks, trestles, and conveyor systems.

8.7 Pile and Drilled-Shaft Foundations

Piles, which are used extensively in the marine environment to carry structural loads through the water column and soft marine deposits to suitable foundation soils or rock, may be designed as either end-bearing or friction-type foundations. End-bearing piles derive their main resistance from the bearing capacity of the hard or dense soils or rock to which the piles are driven. Conversely, friction piles mainly support their load through frictional forces between the piles and the surrounding soils. Drilled shafts are often founded in end bearing on very dense soil or bedrock because of the large vertical loads placed on them. Drilled shafts can also be designed to rely primarily on skin friction where a bearing layer is located at great depth or if the load is too large to be handled strictly by end bearing.

Primary design considerations include (a) the magnitude, direction, and nature of the loading; (b) allowable stresses of the pile or shaft material; (c) the limiting supporting capacity of the soil or rock for single piles, pile groups, or drilled shafts; and (d) the total allowable and differential deflections in the piles or shaft and structural systems.

As a result of the unique environment in which they are used, marine and nearshore deep foundations are subjected to different loadings and conditions from those of upland foundations. Marine piles often are only partially embedded, leaving much of the pile material exposed to severe environmental conditions (Gaythwaite 1981) (see Chapter 3). The design of piles, then, must consider corrosion, marine organism attack, rot, abrasion, impact, and ice damage, as well as cyclic and dynamic loading. Often the presence of soft surface sediments or scour further increases the unsupported pile length, thereby increasing applied moments and decreasing axial capacity.

Marine piles are commonly subjected to substantial uplift forces, lateral loadings, and downdrag forces. The ability of the piles to resist these forces is dependent on the pile material, pile section properties, soil–structure interaction, and soil stress–strain characteristics. Evaluation of ultimate pile capacity and deflection is based on a combination of theory, load test data, and empirical relationships.

The following sections briefly discuss several general pile types that are used in the marine environment, along with some of their advantages and disadvantages (see also Fig. 7-5). A discussion of pile analysis and design considerations for several situations is also presented. Section 7.3 covers structural design considerations of piles. The sections on piles are followed by a discussion of drilled shafts.

Timber Piles

Timber piles, the oldest type of piling, still are commonly used in marine applications. In the past, piles were driven with the bark on for protection against marine borer attack. Today, timber piles are generally pressure-treated with either chromated copper arsenate (CCA) or alkaline copper quaternary (ACQ) for protection (refer to Sections 3.5 and 7.3). Use of each treatment is often regulated, depending on human exposure. Some tropical woods, such as greenheart, are naturally resistant to marine borer attack. The butts of wood piles, however, must be protected from freshwater from precipitation, which can cause severe damage through dry rot and decay (Gaythwaite 1981).

Typical pile capacities range from 15 to 20 tons, with some of the tropical woods extending capacities to 30 tons or more (Quinn 1972). Standard timber pile lengths range from 50 to 65 ft, although lengths of up to 125 ft are possible as special orders.

The natural taper of the pile and a reasonably high pile–soil friction angle make them well suited for friction-type foundations. The same taper, however, detracts from the pile's ability to resist uplift forces.

The end-bearing resistance of the pile may be considerable (Chellis 1961), especially in situations where larger tip diameters are possible. In instances where timber piles are used in end bearing, extreme care must be taken to avoid crushing, cracking, or brooming of the pile tip during driving. A protective tip can be attached to piles where hard driving or obstructions are anticipated.

Prestressed Concrete Piles and Cylinders

Prestressed concrete piles, which are among the most commonly used high-capacity marine piles, can be cast in a variety of shapes, lengths, and diameters. The prestressing strands act as tension elements to resist bending and the large tension stresses induced into the pile during driving (Flemming et al. 1985).

Although they are displacement-type piles, which perform well in friction, prestressed concrete piles often are driven as high-capacity end-bearing foundations. Capacities range up to approximately 200 tons or more, depending on the unbraced length. Delivered lengths range up to about 120 ft. Longer lengths are possible, but shipping and handling become more difficult. A steel plate can be cast at the pile tip for protection of the concrete during driving. If hard driving or obstructions are anticipated, an H-pile "stinger" can be welded to the tip plate (Fig. 8-19).

Fig. 8-19. Steel H-pile "stinger" attached to prestressed concrete pile for hard driving conditions

Large-diameter prestressed concrete cylinders are being used in applications where high vertical as well as lateral capacities are required. Use of the cylinder shape provides moment capacities several times greater than those obtained with solid piles of the same weight (Chellis 1961). Three-hundred-ton, 54-in.-diameter cylinders, 224 ft long, were driven for the construction of a naval pier at Staten Island, New York (Buslov et al. 1988). These large-diameter cylinders also can be used for cantilevered-type retaining walls where satisfactory subsurface conditions exist (see Section 8.5).

The engineer must be careful when determining the length of concrete piles. Although several types of mechanical splices are available (Bruce and Hebert 1974), unanticipated field splices are difficult and expensive to make. Splices should be designed to be located below the mudline whenever possible, and in no case should they end up in the tidal and splash zones. If splicing is anticipated for batter piles, an analysis of how driving stresses are affected by the splice should be performed. Splices may have to be specially designed to take into account eccentric loading caused by the dead weight of the spliced section.

Steel-Pipe Piles

Steel-pipe piles can be used either with or without concrete fill for structural purposes. Marine pipe piles, however, are often concrete filled because of corrosion considerations. Whereas corrosion on the inside of a pipe pile is greatly retarded because the oxygen in a sealed pile is quickly used up, any pinholes caused by external corrosion allow oxygen and seawater to enter, thereby vastly increasing corrosion rates. It may be more economical, however, to increase the pile wall thickness for corrosion protection than to fill the piles with concrete.

Pipe piles are used in both friction and end bearing. Typical load capacities range between 40 and 150 tons (Chellis 1961), with lengths more than 200 ft possible because of the relative ease of field splicing. Pipe piles also provide relatively high section modulus-to-weight and capacity-to-weight ratios, with section properties the same in all directions. Because of the thin wall thickness used in many pipes, drivability should be checked with a wave equation analysis as described in a following section on pile field-testing.

Pipes may be driven either open or close ended. Either case results in a displacement-type pile because of the formation of a soil plug in the open end of the pipe. Water and mud may have to be removed from within the piles before concrete is cast. For this reason, pile tips or end plates are often welded to the pipe before driving. This also allows for internal inspection of the pipe after driving. Use of an open-ended pile or a pile with a concrete plug at the tip permits a grouted anchor to be installed through the pile in cases where large uplift loads must be resisted.

Steel H-Piles

The low-displacement, high-capacity H-pile generally is used where hard or dense soils or bedrock must be penetrated. H-piles typically are designed for loads in the range of 40 to 120 tons (NAVFAC 1986b, DOD 2012), with lengths more than 200 ft possible because of the ease of splicing. These piles are relatively light, are easy to handle, and can withstand high driving stresses. With their low-displacement cross section, H-piles can often be driven close to existing structures without causing damage from heave or vibration. H-piles may become damaged during driving if obstructions are encountered and the tip is not protected with a driving shoe. Unlike pipe piles, H-piles have a weak direction that limits buckling capacity when the pile extends above the mudline. H-piles are also particularly susceptible to corrosion because of their shape and the fact that corrosion takes place from both sides of the web and flanges.

Polymer Resin Piles

The use of composite materials in the piling industry is advancing. Specifically, fiberglass-reinforced polymer (FRP) piles possess many advantages over traditional marine materials such as timber and steel (Lampo et al. 1998b). FRP has recently been used in load-bearing elements as well as fender piles. As more projects are completed with the alternative pile materials and the state of the practice is advanced through research and development, use of FRP piles is likely to increase.

Composite Piles

Composite piles consist of deep foundation elements constructed by combining two different pile types. The use of composite piles is generally dictated by economics and weight considerations. Although the use of precast concrete piles may be the most cost-effective solution, long piles may be difficult to ship and too heavy to handle with normal cranes and marine equipment. Splicing an H-pile or pipe pile onto a concrete pile (Fig. 8-20), however, can substantially reduce weight and result in an overall economical solution. The steel section is placed at the bottom of the composite pile and is designed to be located fully below the mudline to eliminate corrosion concerns.

Drilled Shafts

Drilled shafts, often called caissons, are large-diameter, cast-in-place concrete foundation elements that typically range from 3 to 12 ft in diameter (Fig. 8-21) (O’Neil and Reese 1999). A shaft is constructed by drilling a hole into the subsurface soil and/or bedrock and then backfilling the hole with cast-in-place concrete. On land, the drilled hole can often be kept open with the use of bentonite or polymer

Fig. 8-20. Composite pile consisting of 24-in.-diameter pipe pile welded to a steel plate on the bottom of a 28-in. octagon precast concrete pile

Source: Photo courtesy of Bath Iron Works Corporation

slurries, or a temporary steel casing. Marine caissons, however, often require the use of a permanent steel casing that is vibrated or drilled into place and becomes the concrete form. The steel shaft can also act as a cofferdam when the shaft does not extend above the water level, and it provides a sacrificial protective cover for the concrete. The casing may also be fitted with a cutting head so that it can be spun into the bedrock to allow the drilling of a rock socket.

Once the steel casing has been set, the overburden soil is removed using various tools such as continuous-flight augers, bucket augers, or airlift equipment if the soils are located below a substantial depth of water. Bedrock is removed with

Fig. 8-21. Steel reinforcing cage being installed in a 6-ft-diameter drilled shaft

Source: Photo courtesy of Bath Iron Works Corporation

the use of core barrels, rock augers, roller bits, or down-the-hole air percussion hammers. When the rock socket is complete and the hole properly cleaned, a steel reinforcing cage can be placed inside the casing and the shaft filled with tremied concrete.

Drilled shafts can derive an economic advantage by replacing a multipile group with a single caisson and eliminating the need for a large pile cap to tie the individual piles together. Because of their relatively large cross section and heavy steel cages, drilled shafts can handle substantial vertical and lateral loads. In addition, the massive size of the shaft provides dead weight that can resist uplift forces.

Although large-diameter caissons can provide an efficient and cost-effective foundation design, there can be a significant downside to their use. Shaft construction may require the use of large equipment relative to that required for pile driving operations. The construction of deep marine caissons also requires a high level of skill that few contractors possess. Shaft construction also requires significantly more quality assurance and quality control effort during installation to be successful. Load testing can be expensive and time-consuming, often requiring the use of an expendable "trial shaft." Because one shaft replaces numerous piles, a poorly constructed shaft leaves the designer with few options. The use of large-capacity shafts also provides little redundancy in the structure's design.

Analysis and Evaluation

Pile performance is a function of the pile–structure interaction, pile material and configuration, soil–pile interaction, and soil properties. Evaluation of pile performance generally is based on a combination of theory and empirical relationships. Pile performance may be separated into axial capacity, lateral capacity, and buckling. A discussion of structural design considerations is presented in Section 7.3.

Axial vertical load capacity is dependent on either pile buckling above some point of fixity in the soil or the pile–soil interaction below the mudline. Vertical pile capacity may be evaluated by using static formulas based on (a) the ultimate capacity of the supporting soil or (b) deflections determined from rheological modeling of the pile–soil system using nonlinear springs. Lateral load capacities also may be found by using limit equilibrium methods or deflections based on subgrade reaction theory.

Analysis of the structure can follow two routes. First, the pile can be idealized as a cantilevered column that is fixed at some depth below the mudline (Figs. 7-7 and 8-22). This simplified model is used for analysis of the pile–structure system as

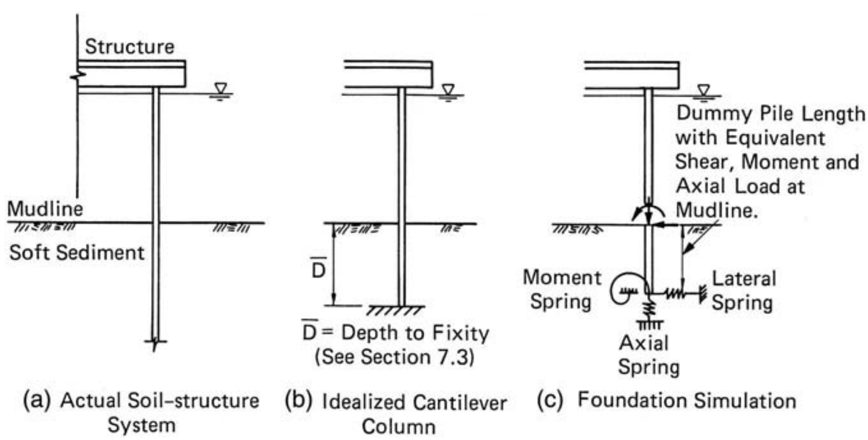


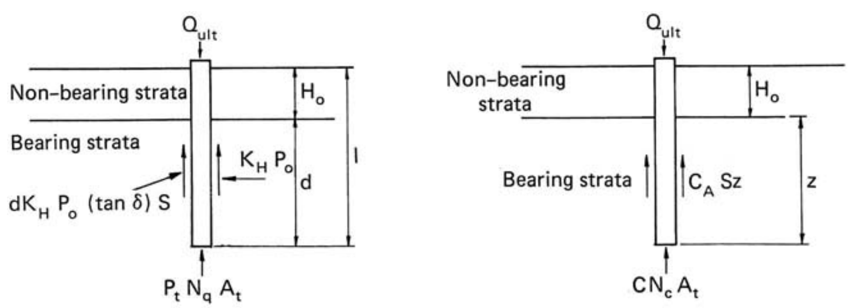
Fig. 8-22. Methods of analysis for waterfront piles

well as for column buckling, but the method results in conservative moments and required pile cross sections below the mudline. Although this analysis may be justified for smaller coastal structures, it is rarely adequate for large structures in deep water. Alternately, the pile–soil system can be modeled along with the structural system to effect a more economical solution. The following sections discuss different methods of analysis for axial and lateral loading conditions.

Axial Load Capacity

The ultimate axial load capacity of a pile may be developed through skin friction, end bearing, or a combination of the two. The theoretical static-load-capacity method of determination of axial capacity is outlined in Fig. 8-23 (Matlock and Reese 1961). In applying the static analysis as indicated in Fig. 8-23 to marine piles, particular attention should be given to friction factors, especially for tension piles.

This classical static analysis typically is used to approximate ultimate capacities based on pile material, configuration, and length. For some marine structures, this type of design approach suffices, but larger, more complicated structural designs often require an accurate description of pile behavior, including capacities with resulting deflections and rotations, for pile–structure compatibility.



$Q_{ult} = dK_H P_o (\tan \delta) S + P_t N_q A_t$

A. Cohesionless

$Q_{ult} = C_A Sz + CN_c A_t$

B. Cohesive

Where:
- K_H = lateral earth pressure coefficient
- P_o = vertical effective stress along pile shaft
- δ = friction angle between pile and soil
- S = pile circumference
- d = depth into bearing soil
- P_t = vertical effective stress at pile tip
- Nq and Nc = bearing capacity factors
- A_t = the area of pile tip
- C_A = soil–pile adhesion
- C = soil cohesion
- z = length of pile in cohesive bearing strata

Fig. 8-23. Static axial pile analysis

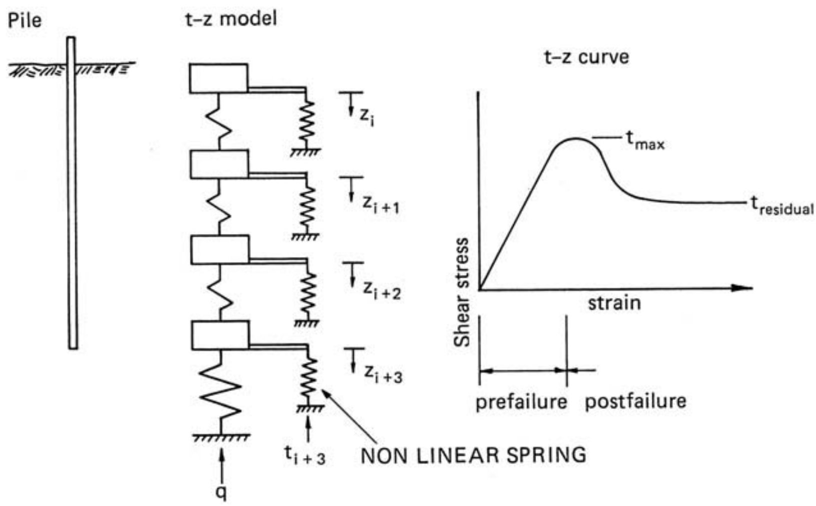


Fig. 8-24. Rheological pile–soil model for *t–z* analysis of vertical pile deformation

Source: Adapted from Focht and Kraft (1986)

Vertical pile deformation may be approximated by using a nonlinear pile–soil system model, referred to as the subgrade reaction or *t–z* method (Focht and Kraft 1986). Fig. 8-24 illustrates this pile–soil model. The pile–soil response along the shaft and tip is represented by *t–z* and *q–z* curves, which describe the relationship between shear stress (t), tip resistance (q), and displacement (z). The analysis of skin friction using this approach is separated into two modes: prefailure, which is based on elasticity, and postfailure, which is based on residual stress-deformation behavior.

At any depth along the pile, the prefailure *t–z* curve is a function of displacement, shear stress, radius of influence, and soil modulus. Postfailure *t–z* analysis is dependent on maximum skin friction, the strain at which maximum skin friction is mobilized, residual skin friction, and strain behavior between maximum and residual stress (Kraft et al. 1981) (Fig. 8-24). Prefailure and postfailure *t–z* curves are developed using displacement equations, laboratory soil testing, modeling, and engineering judgment.

Similarly, a *q–z* curve is developed for the pile tip from the results of instrumented pile load tests, laboratory model tests, or theoretically from the following equation (Kraft et al. 1981):

$$z = q\frac{D(1-\mu)^2}{E_s}I \tag{8-1}$$

where

z = displacement,

q = stress at the pile tip,

D = tip diameter,

E_s = modulus of elasticity of the soil,
μ = Poisson's ratio, and
I = influence coefficient.

Illustration and explanation of the t–z method can be found in Kraft et al. (1981) and Focht and Kraft (1986). Data from the t–z curves for each stratum of soil and the tip q–z curve are input into a finite-difference computer program, along with certain boundary conditions, to model the pile–soil system. The model uses the load displacement curves to simulate foundation response.

Lateral Load Capacity

Lateral load capacity analysis of piles can be separated into two methods based on (a) the ultimate lateral resistance of the soil and (b) the allowable deflection of a loaded pile. In most cases, the lateral load capacity is limited by the tolerable deflection of the structure. The two methods are addressed separately below (Reese and Van Impe 2001).

Classical methods of earth pressure analysis have been applied to ultimate lateral pile capacity analysis. This method now is considered an invalid representation because mobilization of active and passive earth pressures requires excessive movement, which is generally intolerable to most structures. However, a simple static method for determining lateral load capacity was developed by Brinch-Hansen and later modified by Broms (Poulos and Davis 1980). Broms assumed that short piles fail as a result of rotation or translation, depending upon whether the pile head is free or restrained. Long piles were assumed to fail because of the development of one or two plastic hinges along the pile length, again depending upon whether the pile head is free or restrained. The ultimate lateral resistance is based upon the lesser of the lateral capacity of the soil or the maximum pile bending stress (Fig. 8-25).

This empirical method assumes a maximum soil resistance for cohesive soil of nine times the product of the undrained shear strength and pile diameter. For cohesionless soils, the resistance along the pile shaft is taken as three times the Rankine passive pressure, where active pressure along the back of the pile is neglected. A family of curves was developed by Broms to solve the problem, based on the assumptions discussed above but with different boundary conditions. A detailed description of this method and the associated curves is presented in Broms (1965).

There are presently two methods of analyzing lateral pile deflection and associated moments for a given lateral load and pile–structure system: the subgrade reaction theory method and an elastic approach (Poulos 1971).

The subgrade reaction theory is based on the Winkler soil model. The theory assumes that the pile acts as a thin beam with horizontal springs representing discrete soil layers penetrated by the pile (Fig. 8-26). The pile reaction (p) at a point is related to the corresponding deflection (y) at the same point by a spring constant. The spring constant (k) of each spring is the coefficient of subgrade reaction.

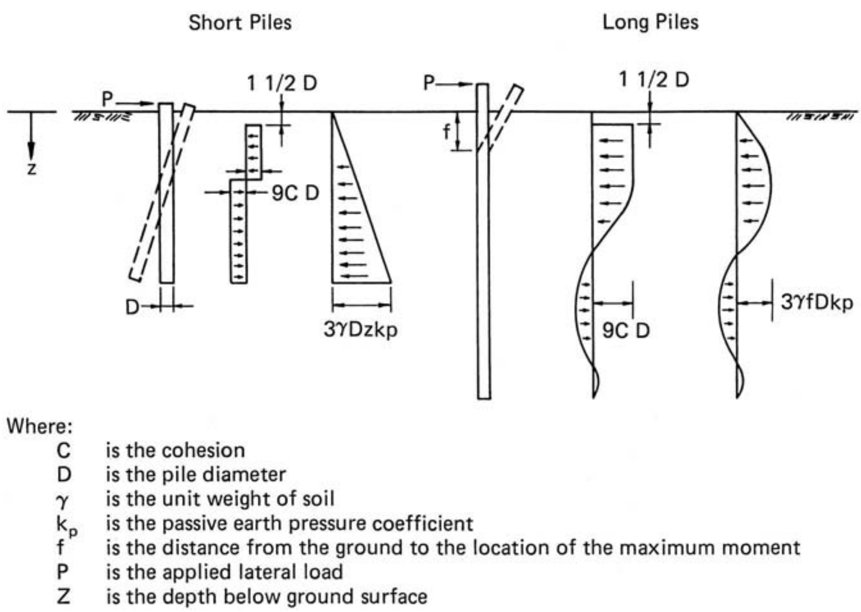


Fig. 8-25. Failure modes for short and long unrestrained piles

Source: Adapted from Broms (1964a, 1965)

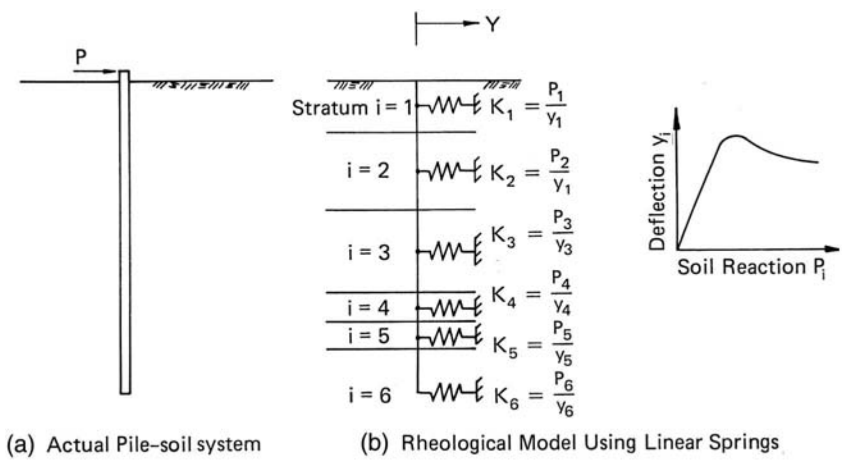


Fig. 8-26. Subgrade reaction theory, *p*–*y* method of analysis for laterally loaded piles

Source: Adapted from Cox and McCann (1986)

A widely used approach to the subgrade reaction model is the *p*–*y* method (Cox and McCann 1986). A solution for the elastic deflection along the length of the pile is obtained by using the differential equation for the deflected shape:

$$E_p I_P \frac{d^4 y}{\partial x^4} + p = 0 \quad (8\text{-}2)$$

where
E_p = pile's modulus of elasticity,
I_p = pile's moment of inertia,
x = depth along the pile,
y = lateral deflection at depth x, and
p = soil reaction.

The soil reaction is given by the equation

$$p = k_s y \quad (8\text{-}3)$$

where k_s is the horizontal modulus of subgrade reaction.

A computer solution for this approach was developed to account for variability of the subgrade reaction with deflection and depth. Actual values of the subgrade reaction usually are based on empirical correlations with other soil properties, but they also may be obtained by using pile load test results and in situ tests, such as those performed with the pressure meter. From these data, representative stress–strain plots (p–y curves) are developed for each soil layer. Consequently, a relative stiffness factor is developed for each element along the beam based on the p–y curves. The deflection of the beam is modeled through trial and error until the deflection and resistance patterns of the beam agree as closely as possible with the resistance–deflection relations of the soils (Reese and Van Impe 2001).

Determination of Depth to Fixity of Partially Embedded Piles

When partially embedded piles penetrate a surface stratum of weak marine deposits, only moderate lateral pile support can be expected from these soils. Intuitive reasoning suggests that the amount of lateral resistance should increase with depth to full lateral restraint in competent soils. As discussed previously, an idealized model of this condition was developed to simplify the analysis of the pile–structure system and to determine an effective unsupported pile length for column buckling analysis. The idealized pile–soil reaction is represented by a cantilevered pile of equivalent stiffness, fixed at some depth below the mudline. The depth between the mudline and the point of fixity is called the depth to fixity ($\overline{D}$) (Fig. 7-7), a term that was introduced in Section 7.3 along with associated Eqs. (7-1) and (7-2). Depth to fixity, as calculated by these equations, is equal to one-half of the critical length as determined in lateral pile deflection analysis using subgrade reaction theory. The critical length is defined as the length beyond which the pile behaves as an infinitely long beam (Poulos and Davis 1980). Therefore, Eqs. (7-1) and (7-2) actually are modifications of equations developed from subgrade reaction theory as applied to laterally loaded piles.

For Eqs. (7-1) and (7-2), soil resistance to lateral movement is represented by a subgrade reaction modulus (k_s) when k_s is constant with depth, or by a coefficient of subgrade modulus (n_h) when k_s varies with depth. Determination of these soil properties is done by one of the following methods: full-scale lateral pile load tests, plate load tests, or empirical correlations with other soil properties. Several empirical values are presented in Section 7.3 and in NAVFAC (1986b), DOD (2012), and Matlock and Reese (1961).

In summation, static analysis methods for determining pile–soil capacity and the depth of fixity method for analyzing pile buckling generally are used in the design of most nearshore structures. The analysis then is verified with full-scale pile load tests. For larger, more complicated structures, more sophisticated analysis is warranted for accurate simulation of the foundation response. These methods generally are performed using computer solutions with a combination of empirical information and test data.

Uplift Resistance

Because of the existence of relatively large uplift and lateral loads common to marine structures, piles often must be designed for tension as well as compression. Uplift capacities typically are calculated in the same way as shaft friction resistance for compression piles. Conservative values for the coefficient of lateral earth pressure for cohesionless soils and adhesion for cohesive soils should be used (NAVFAC 1986b, DOD 2012).

When insufficient tension resistance is available through shaft friction, several options are available. The simplest method of increasing uplift resistance is to drive the pile deeper than normally required for compression, thereby increasing the shaft length. This method is not always possible because of shallow rock or hard driving conditions. Alternatively, additional piles can be driven in order to increase uplift resistance. If several piles are driven close together, the uplift may be limited by group action (Teng 1962). The addition of lagging may be useful for piles that are driven through soft soil (Chellis 1961), where the lagging consists of bolting or welding additional sections to the pile shaft to effectively grab surrounding soil and mobilize its dead weight for uplift resistance. If large tension forces must be resisted, grouted anchors can be drilled through open-ended pipe piles.

A recent pile product has been introduced to the market that allows for increased uplift capacity with relatively short installed pile lengths by SPIN FIN pile. SPIN FIN piles are a deep foundation alternative that can resist the combination of compression and tension loads typically associated with marine berthing and mooring structures. This patented pile design consists of a pipe pile with steel fins welded around the pile perimeter near the pile tip (Fig. 8-27). Rather than being vertical, the fins are slightly wrapped around the pile perimeter in a gentle screwlike configuration. This pile is driven with standard pile driving equipment, but it rotates into the ground as it is driven. The fins modify the pile behavior under loading, resulting in the development of both tension and compression resistance significantly greater than that of a traditional nonfinned pipe pile. The net result is larger

Fig. 8-27. Bottom of SPIN FIN pipe pile

Source: Photo courtesy of the Woods Hole, Martha's Vineyard, and Nantucket (Massachusetts) Steamship Authority

capacities using shorter installed pile lengths (Campbell 1986). This result is particularly important for hard driving conditions or where bedrock is shallow and significant tension resistance is required.

Batter Piles

Batter piles, sometimes referred to as *raker piles*, often are used to resist lateral loads applied to the structure. These piles usually are driven on batter angles ranging from 1 to 5 horizontal to 12 vertical. Batters shallower than 5 on 12 become difficult to obtain because of pile driving equipment limitations. In addition, shallow batter angles can lead to large bending stresses caused by the weight of the pile.

The pile heads must be attached to the structure or other piles in order to adequately transfer loads. If the attachment is assumed to be a pinned connection, the lateral load results in an axial load in the sloped pile. These piles generally are analyzed in the same way as axially loaded vertical piles.

Because a batter pile transmits load axially, any horizontal load has a resulting vertical component that must be resisted in order to satisfy equilibrium. This vertical component of load may be resisted by the dead weight of the pile cap and structure or by tension resistance in other piles.

Batter piles often are driven in opposite directions and coupled, so that when one pile acts in compression, the other acts in tension. Compression loads are resisted by a combination of shaft friction and end bearing. The tension load, however, can be resisted only by friction. When insufficient tension capacity is available through shaft friction of a single pile, additional piles may be required. Also, other methods of increasing tension resistance, as discussed in the previous section, may be used. Batter piles are particularly susceptible to downdrag forces (see below) when compressible subsurface soils experience settlement. Because of the sloped configuration of the pile, large bending stresses may be introduced into the pile shaft.

Downdrag

Downdrag, or negative skin friction, is developed when all or a portion of the overburden soil settles relative to the pile. The force mechanism is the same as normally calculated for shaft resistance, but it acts in the opposite direction. The downdrag forces are developed not only in the layer that experiences settlement but also from all the natural soil or fill above this layer.

The effect of these often substantial loads may be pile settlement if the remaining shaft friction and bearing capacity is exceeded, or pile crushing if yield stress is reached before soil or rock failure.

Negative skin friction can be decreased by preaugering before pile driving, thereby reducing the friction coefficient or horizontal stress along the shaft. In some cases, the preaugered hole can be filled with a bentonite slurry to eliminate negative skin friction in the areas subject to downdrag forces. The bentonite might be used only temporarily until settlement is complete, at which point it is displaced by backfilling with coarse gravel or crushed stone. Still another method used to reduce negative skin friction is to cover the pile with a bituminous coating (Baligh et al. 1978). The coating tends to lubricate the pile, thereby reducing the detrimental effects of settlement.

Driving Criteria and Field Testing

Once a pile has been designed, the contractor needs to know what equipment to use to drive the pile, and the engineer needs a way to check that the driven pile actually attains the anticipated capacity. There are several ways to determine the driving criteria, based on the pile type, pile driving equipment, and subsurface conditions. Performance of a full-scale load test is generally the method of determining the actual pile capacity after the pile has been driven (Fuller 1983).

Dynamic pile-driving formulas have been used for many years to determine the static capacity of a pile based on the energy delivered to the pile and the "set" or pile movement. Developed in the 1930s, the *Engineering News-Record* (ENR) formula has probably been the most widely used throughout the years, although currently it has been mostly supplanted by more contemporary methods.

Numerous other more sophisticated dynamic driving equations have been developed in recent years. Most of them have the same general form and do not necessarily produce better results. Any driving formula should be considered appropriate and should only be used for low-capacity piles for noncritical structures. Most building codes only allow the use of such formulas for pile capacities less than about 40 tons.

A more accurate analysis can be performed using a one-dimensional computer model based on a wave equation analysis of pile driving, commonly referred to as a WEAP analysis (Goble et al. 1975). The WEAP analysis models the hammer energy delivered to the pile head as a stress wave that travels down the pile shaft. The pile is modeled as a series of discrete masses and soil springs based on the pile material, soil type, and soil density. The program gives a blow count criterion to which the pile should be driven in order to attain the ultimate capacity (typically two times the design capacity). The WEAP analysis requires considerable experience in determining the input values for the hammer and soil. The program also provides maximum tension and compression stresses in the pile caused by driving. This maximum is important in selecting a foundation element that can be driven to the desired capacity without damaging it.

Another method used to monitor the actual pile driving operation is with the use of a pile driving analyzer (PDA). The PDA monitors the pile as it is driven through the use of accelerometers and force transducers that are attached to the pile shaft near the pile head. The data from the instrumentation is recorded and analyzed with a digital processor. The Case Method (Goble et al. 1975), which is a wave analysis similar to the WEAP analysis, is used to analyze the pile driving operation using actual field data rather than assumed values. The PDA provides information on hammer performance (actual energy transmitted to the pile), capacity, maximum pile stresses, and pile integrity. The PDA is often specified to be used to determine pile capacities in lieu of a load test. This specification is common for marine projects because of the high cost of setting up a full-scale load test over the water for large-capacity piles. If a load test is not performed, several piles should be tested with the PDA, and a higher factor of safety might be used. The Federal Highway Administration recommends a factor of safety of 2.25 for PDA-tested piles (FHWA 2010).

PDA results are highly dependent on certain soil parameters that are entered into the program. Determination of these parameters takes experience with the local soil types and characteristics in order to accurately predict pile capacity. In addition, if a pile hammer energy is insufficient to move the pile tip, the PDA cannot analyze the pile capacity. For this reason, WEAP analysis is often used to initially size the pile hammer, based on the hammer and soil characteristics, followed by use of the PDA during test pile installation.

Full-scale load testing may be required by building codes for piles with design capacities over approximately 40 to 50 tons. A typical compression load test is performed by jacking the pile against dead weight stacked on a frame constructed over the pile. Alternately, the pile can be jacked against a beam that resists movement through the use of rock anchors or tension piles. In either case, performing this type of test over the water is difficult, costly, and time-consuming. As the load is jacked into the pipe, up to twice the design load, movement is monitored and plotted against load. The pile is subsequently unloaded, and the data are analyzed against one of several criteria in order to determine the safe design load capacity (ASTM 2002). Load tests can also be run to determine allowable tension and lateral capacities (ASTM 2002).

Drilled shafts also require load testing in order to confirm the assumed design values for the soil and rock and to ensure proper construction techniques (O'Neil and Reese 1999). Load testing of shafts is somewhat more difficult because of the large load capacities and the difficulty of providing enough dead load or rock anchors to resist the required loading. One method of testing a shaft is to place a hydraulic jack, known as an Osterberg cell, at the bottom of the rock socket before concreting. The Osterberg cell is jacked against the completed shaft using the dead weight, soil friction, and bond stress between the shaft and the side of the rock socket as resistance. With the use of strain gauges at the cell and on reinforcing steel within the shaft, the resistance along the shaft, bond along the rock socket, and bearing capacity of the underlying bedrock can be determined. If a production shaft is used for the testing program, the hydraulic fluid is flushed out of the cell and replaced with high-strength grout when the test is complete. Often, however, a nonproduction "trial shaft" is constructed. The trial shaft is sometimes constructed with a smaller diameter to decrease cost, reduce the amount of materials, and allow for easier removal if required. The same information can be obtained from the smaller diameter shaft.

Another way to test caisson-type foundations is with the use of a Statnamic load test (O'Neil and Reese 1999). The Statnamic system is capable of delivering large loads to a foundation element by launching a reaction mass upward away from the top of the shaft by igniting a propellant fuel placed between the reaction weight and a load cell on top of the shaft. The reaction weight is contained within a metal sheath to control its movement. As the reaction weight is accelerated upward, load is applied and the shaft is monitored with instrumentation. The Statnamic test also has the advantage that it can easily be applied to a production shaft that was not originally planned to be tested. The same method can also be used to apply lateral test loads.

8.8 Dry Docks

There are several types of dry dock and ship lift facilities, including marine railways, vertical lifts, floating docks, mobile straddle lifts, and basin or graving type docks (refer to Chapter 10). These facilities must handle heavy vertical loads as well as appreciable lateral loads caused by wind and, in some cases, wave and current forces. Foundation types are varied and depend on the type of dock, facility configuration, and subsurface conditions.

Geotechnical Considerations

Marine railways, vertical ship lifts, straddle lift piers, and floating dry dock mooring dolphins typically are supported by pile or drilled shaft foundations. In certain situations, many of these dry dock systems are used in conjunction with solid-fill structures. Shallow foundations also are commonly used for transfer systems where competent soils exist, and in some cases are used for marine railways.

Graving docks provide some of the most interesting and challenging geotechnical design problems. Graving docks can be constructed within a cofferdam built out into the water, or they can be constructed inland to be finally opened to the water by excavating and dredging in front of the gate.

Dock walls must be designed for lateral earth pressures as well as hydrostatic forces when the dock is dewatered. The walls generally are constructed of reinforced concrete. However, if a cofferdam is used, it may be incorporated into the final structure (Sorota and Kinner 1981a, b). Anchored slurry or diaphragm-type walls (Xanthakos 1979), steel sheet piling, or concrete caissons also can be used in conjunction with a concrete floor in certain situations (see Section 10.2).

The dock floor must be designed to resist hydrostatic pressures, and the whole dock must resist overall buoyancy. This requirement can be accomplished by constructing a heavily reinforced concrete floor to resist the induced bending stresses. The floor dead weight along with the weight of the walls and soil above the extended base can be used to resist uplift (Fig. 8-28). An approach that usually is more cost-effective than this, however, is to use grouted anchors to reduce spans, resist buoyancy forces, and decrease floor thickness (Hanna 1982). Vibrated, jetted, or augered anchors have also been used for the same reason (Hanna 1982).

Grouted anchors, embedded plates, and augered anchors derive their resistance from the dead weight of soil or rock within the cone of influence (Fig. 8-29). The size or half angle of the cone is dependent on the friction angle of the soil, or the fracturing, bedding, and degree of weathering of the rock. Half angles of the cone

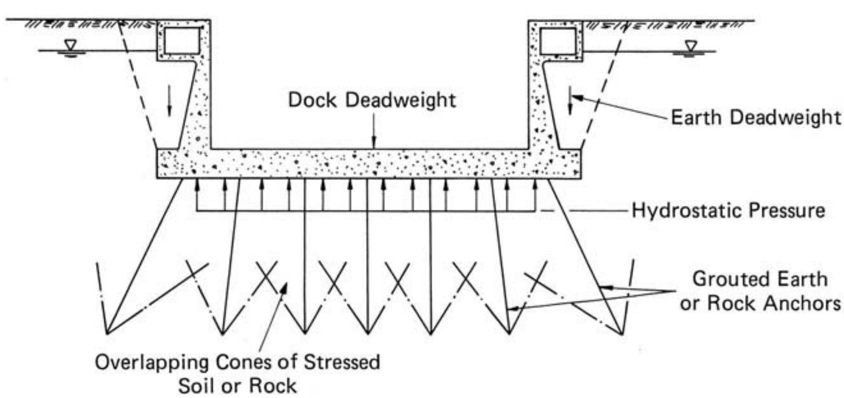


Fig. 8-28. Basin dry dock with grouted tie-down anchors

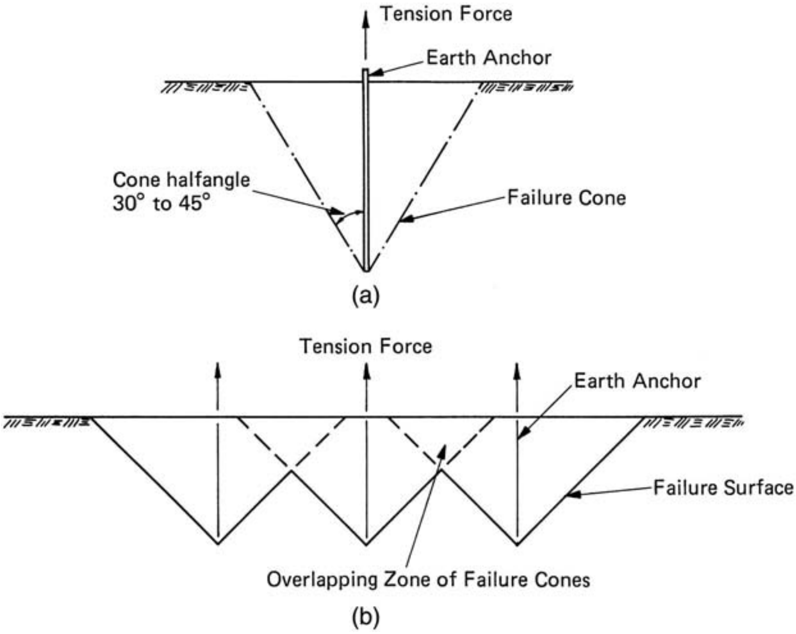


Fig. 8-29. Cone of influence for grouted earth anchors: (a) single anchor; (b) overlapping cones for multiple anchors

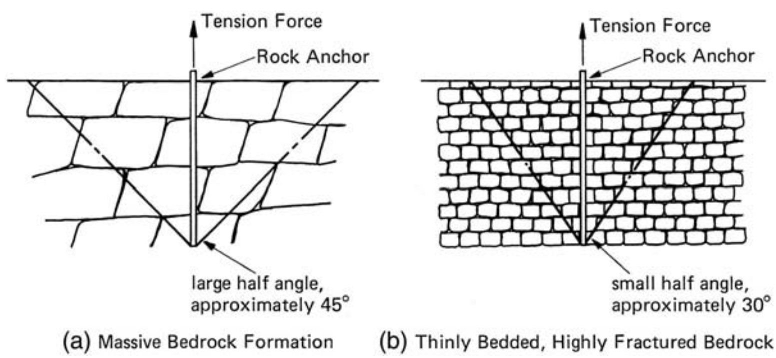


Fig. 8-30. Influence of rock fracturing and bedding on grouted anchor cone of influence

typically range from 30° to 45°, but may be less, depending on bedding angles (Hanna 1982) (Fig. 8-30). In any case, where anchors are closely spaced, an allowance must be made for overlapping of the cones. When calculating the soil or rock deadweight resistance, buoyant unit weights should be used. In addition to the deadweight resistance, two other modes of failure must be investigated. The bond resistance between the grout and the surrounding soil or rock and the bond resistance between the anchor rod and grout must be checked. The length of the anchor is determined by whichever mode of failure proves to be critical.

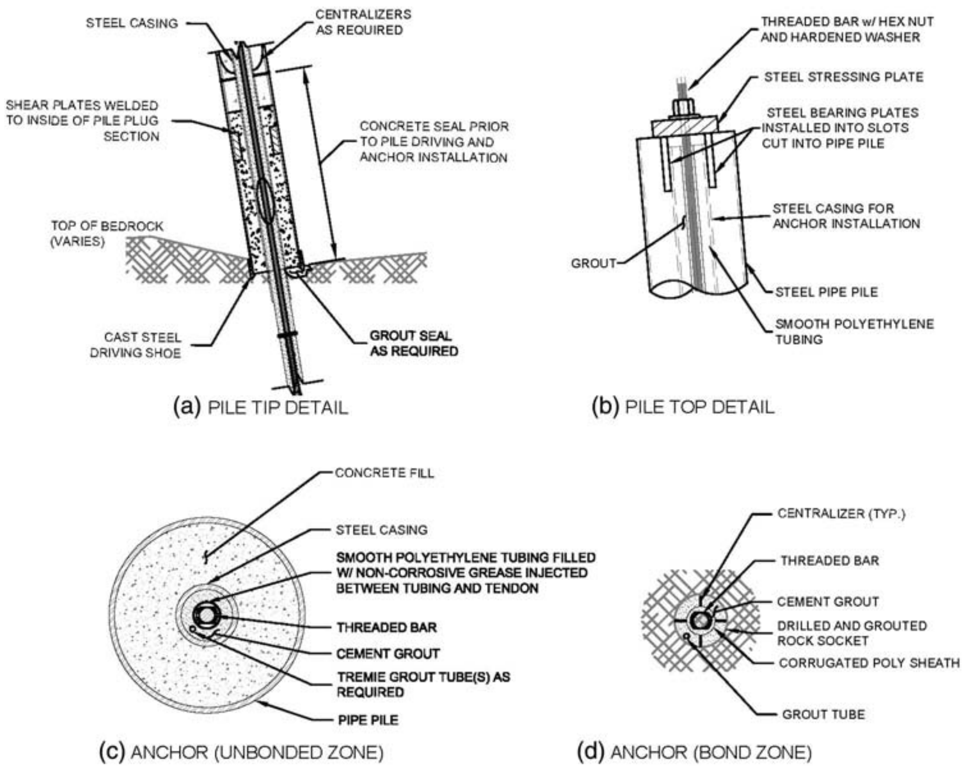


Fig. 8-31. Typical rock anchor details

Fig. 8-31 shows typical grouted anchor details. Grouted anchors can also be used for rehabilitation of aging dry docks that may have problems with wall movement, failing underdrain systems, or other buoyancy problems.

Because of the cyclic buoyancy nature of a graving dock operation, care must be taken in the design of the anchors. Prestressing forces must be greater than the buoyancy loading from dewatering in order to maintain contact pressure between the ground and the dock. Although the mechanisms are not well understood, repeated loading reduces anchor capacity (Hanna 1982). Bember and Kupfennan (1975) suggest a factor of safety of 2.5 for ultimate static load compared to the peak cyclic load. More extensive information on the design of tension anchors is provided in Hanna (1982) and Schnabel (1982).

Tension piles can provide resistance to uplift and are advantageous when a pile foundation already must transfer loads across a weak or compressible layer. The calculation of anchor pile resistance (tension piles) is discussed in the previous section. Where closely spaced piles are used, single-pile as well as group action must be checked to determine limiting values.

As an alternative to designing the dock to resist hydrostatic pressures and buoyancy forces, a pumped pressure-relieving subdrain system can be installed.

Hydrostatic pressures on both the floors and walls may be relieved by such a system. A pumped system located above cohesive soils, however, can lead to consolidation of the layer and settlement of the dock.

8.9 Site Improvement: Methods and Materials

Site improvement or ground modification can be an important phase in the development of waterfront sites. Several methods and materials can be used to densify loose soils, reduce postconstruction settlement, and increase stability. Some of these methods also can be used to densify backfill materials, to increase soil strength and friction angle, and to reduce the possibility of liquefaction potential during earthquakes (Elias et al. 2000). In addition, there are several relatively new products, as well as products whose applications are new to geotechnical and marine engineering, that can assist the engineer in the design of waterfront structures.

Vibroflotation and Vibro-Replacement/Stone Columns

Vibroflotation and vibro-replacement are two commonly used methods for the improvement of deep, soft, or loose soils (Brown 1977, Harder et al. 1984). *Vibroflotation* is performed by using a vibroflot probe, which vibrates as well as uses water jets to penetrate into loose granular soils as it is lowered by a crane. When the appropriate depth is reached, the forward water jets are turned off, and the horizontal vibrator in the probe tip compacts the soil as the equipment is extracted in 2-ft increments. Granular backfill material is continually fed into the void left by the probe, and the vibration compacts the backfill as well as the surrounding material (see Figs. 8-32 and 8-33). The operation leaves a column of densely compacted material approximately 7 to 9 ft in diameter.

Vibro-replacement, or stone columns, is a variation on the vibroflotation technique used for in situ stabilization of cohesive organic soils or densification and strengthening of loose cohesionless materials (Barksdale and Bachus 1983). Stone columns are composed of coarse gravel or crushed stone that is compacted as it is placed. A vibroflot probe is used to open the hole for the column, and the stone is either dumped in from the surface or placed directly at the probe tip through a chute. Each load of stone then is compacted by the weight and vibration of the probe as the backfill material is pushed laterally into the surrounding soils. The column diameter depends on existing subsurface conditions.

Stone columns typically are placed on a grid with a center-to-center spacing ranging from 3 to 10 ft. A series of columns can be placed under a new structure to improve bearing capacity and allow for the use of a shallow foundation, as opposed to a deep foundation. The use of stone columns can be less expensive than a pile foundation and can aid in the drainage and consolidation of subsurface layers. This help can be important when placing fill at sites that have underlying deposits of

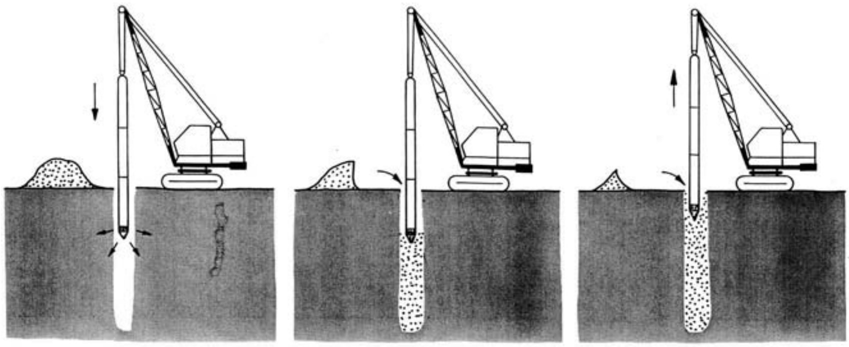


Fig. 8-32. Schematic of vibro-compaction and vibro-replacement technique

Source: Courtesy of Hayward-Baker

Fig. 8-33. Stone column vibro-replacement equipment

Source: Photo courtesy of Hayward-Baker

cohesive soils. Stone columns can also provide a conduit for excess pore water pressures to escape during earthquake events, thereby reducing the risk of liquefaction.

As mentioned in Section 8.4, vibro-replacement can be used to increase the stability of slopes (Dobson 1986). The strength and the friction angle of the subsurface soils are improved by the placement of stone columns. Also, the columns can

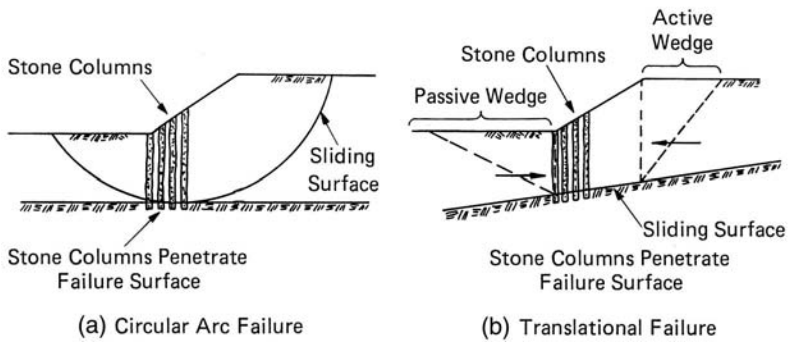


Fig. 8-34. Use of stone columns or rammed aggregate piers to increase slope stability

Source: Adapted from Dobson (1986)

penetrate through possible slip surfaces, thereby increasing factors of safety against sliding. Fig. 8-34 presents a schematic of the use of stone columns to aid slope stability.

Rammed Aggregate Piers

Rammed aggregate piers are a ground improvement method commonly used to reinforce soft or loose soils (Lawton and Fox 1994, Moser et al. 1999, Fox and Edil 2000, Fox and Lein 2001, White et al. 1963). The rammed aggregate elements are installed by drilling 24-in. to 36-in. diameter holes with an excavator-mounted auger and ramming thin lifts of well-graded aggregate within the holes to form stiff, high-density aggregate piers, as shown in Fig. 8-35. The drilled holes typically extend from 10 to 30 ft below grade. The first lift of aggregate forms a bulb below the bottoms of the piers, thereby prestressing and prestraining the soils to a depth equal

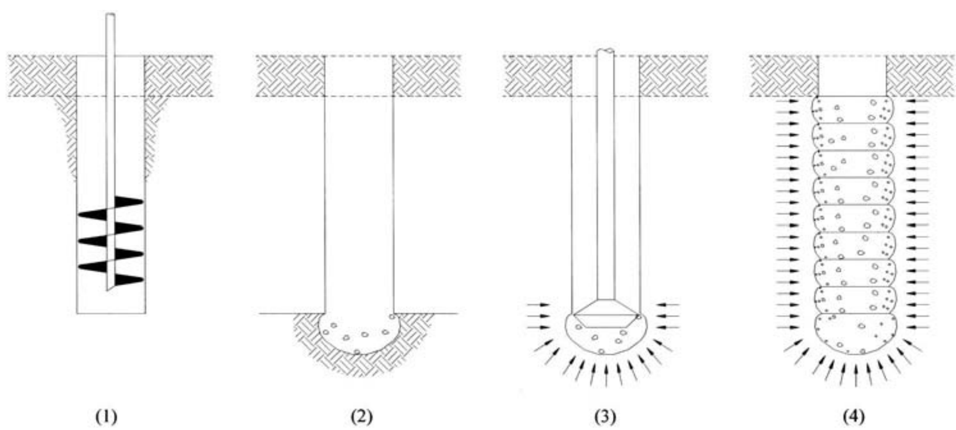


Fig. 8-35. Rammed aggregate pier soil reinforcement construction process

Source: Photo courtesy of Geopier Foundation Company

Fig. 8-36. Rammed aggregate pier installation

Source: Photo courtesy of Geopier Foundation Company

to at least one pier diameter below drill depths. Subsequent lifts are typically about 12 in. thick. Ramming takes place with a high-energy beveled tamper that both densifies the aggregate and forces the aggregate laterally into the sidewalls of the hole. This action increases the lateral stress in the surrounding soil, thereby further stiffening the stabilized composite soil mass. The construction process is shown in Fig. 8-36.

The augured holes are generally drilled uncased in cohesive soils. In granular soils, a hollow stem tamper is used to open the hole. Open graded aggregate is fed to the bottom of the tamper from a chute. The tamper is alternatively raised to allow the crushed stone to fall into the hole and then lowered to compact the pier. Grout can be added to the column to stiffen it in soft and organic soils to avoid bulging of the pier.

The result of rammed aggregate pier installation is a significant strengthening and stiffening of subsurface soils that then support high-bearing-capacity footings and control settlement. Rammed aggregate elements are installed beneath shallow footings or in grid patterns for support of floor slabs or area fills. The rammed aggregate pier system is designed to reinforce the soil where the stresses are highest immediately beneath the foundations. As a result, overall foundation settlements are controlled by significantly reducing compression of the highly stressed near-surface soils.

Rammed aggregate pier soil reinforcing elements are typically used to provide support of shallow foundations and floor slabs for ancillary structures in the backlands of marine facilities. With the rapid installation process averaging 40 to 60 elements per day, rammed aggregate pier elements can provide both cost and time savings compared to deep foundations. Rammed aggregate piers can also be used to increase the stability of slopes, as shown in Fig. 8-35.

Vibrating Probe Compaction

The vibrating probe compaction technique uses a specially fabricated, open-end, large-diameter pipe pile that is vibrated into loose sand and gravel (Anderson 1974). Vibrations are delivered to the pile by a vibratory pile-driving hammer and cause densification of loose materials as the pile is placed and extracted. Raising and lowering of the vibrating pile can take place several times at each location until the desired density is obtained (see Fig. 8-37). Brown and Glenn (1976) describes a comparison of the vibro-compaction and vibrating probe techniques based on full-scale field tests. The project consisted of the densification of hydraulically placed

Fig. 8-37. Vibrating probe compaction used to densify loose sand in cofferdam bulkhead to eliminate possible failure caused by earthquake-induced liquefaction

Source: Photo courtesy of Bath Iron Works Corporation backfill for the construction of a graving dry dock. Vibrating probe compaction can be performed upland or below standing water.

Deep Dynamic Compaction

Deep dynamic compaction is a landside method that is used to compact loose granular materials. The method consists of raising and dropping a large mass, usually a concrete or steel block, onto the ground surface. The induced vibrations cause densification of the subsurface material. Different energies are easily obtained by increasing or decreasing either the mass of the falling block or the free-fall height (Fig. 8-38).

Fig. 8-38. Deep dynamic compaction

As in the vibratory methods described above, an in situ test program is performed in the field and checked with the SPT or cone penetration test to determine the grid spacing, mass of the block, free-fall height, and number of blows required at each location. A particular advantage of deep dynamic compaction is that it can be used in areas that have been filled with boulder, rock, or demolition fill, a situation that precludes the use of most other methods. Unfortunately, testing of compaction performance in this type of material is difficult to impossible, although measurement of the average total settlement gives an indication of the void ratio reduction. Deep dynamic compaction produces relatively large vibrations, compared to other vibratory methods, which can cause problems with nearby facilities.

All of the above site improvement methods are generally carried out in grid-type fashion over the entire area to be compacted. The grid spacing usually is determined by field testing before development of the compaction program. Additional probes or drops can be completed at particular locations to pick up large column or other concentrated loads. Results are verified by performing SPT or cone penetration tests between grid locations after densification (Welsh 1986). The depths of the divots formed by the probes or drops are monitored during the ground improvement process to look for softer or weaker areas. These areas can be retreated with additional passes of probes or drops at a closer spacing.

Preloading/Surcharging

An increase in site grade or additional loads caused by proposed structures can result in unacceptable settlements where underlying compressible layers are present. However, this problem can be ameliorated by preloading/surcharging, a method of preconsolidating compressible soils to a stress level greater than the stresses caused by new construction loadings (Stamatopoulos and Kotzias 1983). The method generally is performed by stockpiling fill on the site until the underlying soils have consolidated. Other preloading/surcharging methods, including temporarily storing bulk products such as coal, scrap steel, or iron ore onsite, or constructing lined basins in order to flood large areas, also have been successful.

Preloading/surcharging can be a time-consuming process, depending on the permeability of the soil and the thickness of the compressible stratum. A thick deposit of low-permeability soil can require an excessive time span to modify the site. The use of closely spaced vertical sand drains or wick drains can speed the process considerably, however, by providing a relatively short drainage path for pore pressure dissipation.

Vertical Wick and Sand Drains

Wick drains and sand drains are vertically installed drainage elements used to hasten consolidation of deep deposits of cohesive soils (Fig. 8-39). Sand drains are columns

Fig. 8-39. Typical wick drain installation equipment

Source: Photo courtesy of TerraSystems, Inc.

of sand that are installed by driving a pipe pile fitted with a trapdoor bottom plate into the ground. The pipe is subsequently filled with sand that stays in the ground as the pipe is extracted. Wick drains are corrugated plastic strips covered with filter fabric that are pushed into the ground with a specially designed mandrel. Wick drains have been used extensively to decrease the time required for drainage of pore water from deep deposits of cohesive materials (Hansbo 1979). The vertical drains are used to accelerate consolidation of fine-grained materials and avoid a buildup of excess pore water pressure that can lead to slope stability problems. Closely spaced wick drains provide a short horizontal drainage path for pore water to travel to where it is conveyed to the surface and carried away.

In the few years that wick drains have been available, they have received wide acceptance and have almost totally replaced the use of sand drains. They are cleaner, easier, and faster to install and are more reliable, efficient, and cost-effective than sand drains (Morrison 1981). It should be noted, however, that in certain situations the filter fabric can become smeared with fine-grained soils that reduce the effectiveness of the method.

Lightweight Fill

Lightweight aggregate, commonly used in the production of lightweight concrete, currently is being used as a geotechnical backfill material. Its high strength-to-weight ratio can be used to solve stability, settlement, and high lateral earth pressure problems associated with many waterfront structures.

Rotary-kiln-produced expanded shale aggregate has a number of important properties that make its use as a lightweight backfill material viable. The angle of internal friction is in the range of 40° to 45°, and compacted dry unit weights below 65 lb/ft^3 are possible (Childs et al. 1983). Individual particles have fairly high abrasion resistance and do not exhibit appreciable breakdown in the field. The material also can be supplied in a number of standard gradations.

Though lightweight fill is more expensive than granular backfill material, it can lead to an economical solution by allowing the use of lighter structural members for new construction. Lightweight backfill also may be used to relieve lateral earth pressures on existing distressed structures. Not only has lightweight aggregate been used successfully for numerous small rehabilitation projects, but also its use on large-scale projects has proved economical (Carchedi and Porter 1983).

Low-density cellular concrete recently has been used to reduce loads on existing waterfront structures (Palermo 1985). The material is aerated before being pumped in a slurry consistency, and resultant unit weights are on the order of 36 lb/ft^3. Buoyancy of the material can be an issue if closed-cell concrete is used. Recently, an open-cell, low-density concrete has been developed that saturates when flooded rather than floats.

Mechanically Stabilized Earth

Mechanically stabilized earth (MSE) is another relatively new concept in foundation engineering. Originally developed for a vertical retaining wall system (Koerner 1997), it has proved useful in many other foundation applications. MSE retaining walls are built with individual interlocking members attached to a series of rows of reinforcing strips or geosynthetic grids. The strips or grids reinforce the soil by acting as a series of tension members, reducing lateral earth pressure to such an extent that only lightweight face panels or blocks are required to prevent the loss of fill.

The use of reinforced earth in the marine environment has been reasonably successful (Ingold 1982, Munfakh 1985) (Fig. 8-40). Construction can be somewhat difficult, however, because of the nature of the environment, especially where deep water or large tidal fluctuations exist. The use of filter fabrics or face unit sealants usually is required to prevent migration of fines from the backfill. Concerns about the corrosion of metal reinforcing strips have been addressed with coatings and the recent use of geosynthetic grids.

The concept of soil reinforcement also can be applied to the design of embankments and fills over soft ground. The reinforcing material allows the use of steeper slopes and reduces the possibility of stability failure. Site fills layered with inclusions of geotextiles or geogrids can sustain higher surface loads from cranes and other marine facility equipment.

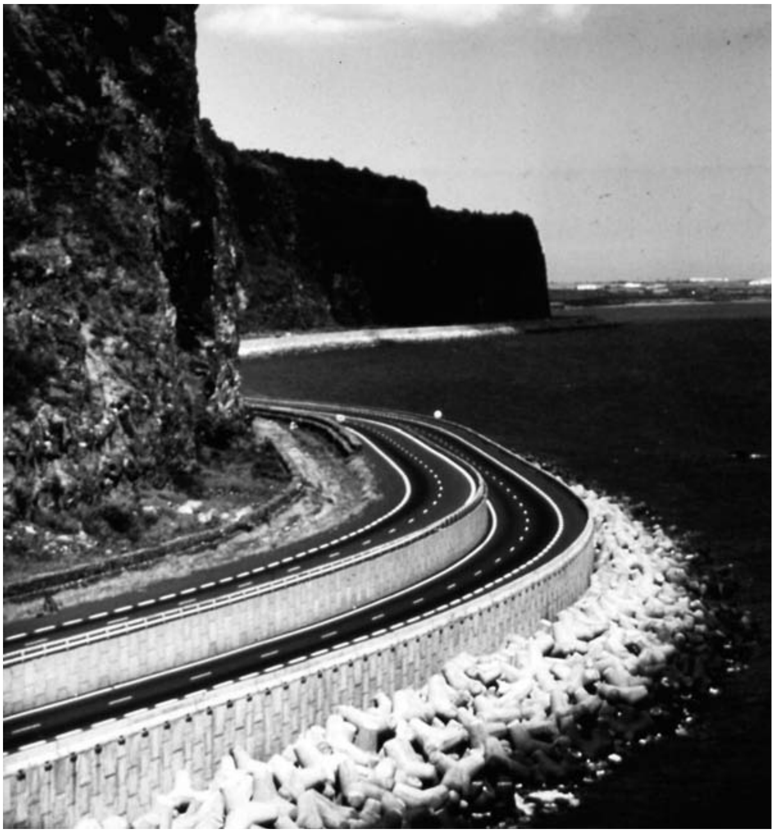


Fig. 8-40. Use of reinforced earth wall in the marine environment

Source: Photo courtesy of Reinforced Earth Company

Fabrics and Filter Materials

The use of fabrics in geotechnical engineering has boomed in recent years. Numerous types of fabrics are being used for filters, impermeable barriers, reinforced earth, wick drains, drainage mats, and erosion control.

In waterfront engineering, the use of fabrics has been mainly for filters and erosion control. Normally, filters for granular soils used in coastal structures are made up of graded layers of gravel and stone (PIANC 1992). Materials in the proper gradation are often of limited availability or costly, and proper placement is often time-consuming and difficult to control. Woven fabrics have been used successfully in a number of coastal structures to alleviate filter problems and preclude the possibility of structural failures caused by leaching and erosion of construction materials (Dunham and Barrett 1974).

Fabrics have been widely used to distribute loads from equipment, fills, and stockpiled materials. When placed over soft or organic soils, they also can help to control mud waves from subsequent filling or capping operations.

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Floating Port Structures

Floating structures are prominent features in many ports, serving as floating docks for small-craft berthing and sometimes as piers or wharves for larger oceangoing vessels. There are also floating dry docks, breakwaters, mooring and navigation buoys, camels and separators, containment booms, and plants and equipment of a wide variety. This chapter is primarily concerned with basic design principles that apply in general to all floating-structure types, with particular regard to floating-pier applications. Basic principles of buoyancy, stability, motion response, and certain aspects of structural design are reviewed. Mooring and anchoring systems are of major importance in floating-pier design, so their basic design principles are presented. Means of access to floating piers; ancillary systems such as ballast control, pumping, and flooding; and miscellaneous design features are reviewed. The final section of this chapter is devoted to floating docks for marinas and small-craft facilities.

9.1 Structure Types and Applications

This section provides an overview of basic configurations and applications of floating structures commonly used in the berthing and mooring of vessels. Floating dry docks and floating caisson gates that form the closure of basin-type dry docks, which also are common port floating structures, are discussed in Chapter 10.

Floating Piers and Platforms

Floating piers are usually made in one of three basic hull configurations, as illustrated in Fig. 9-1: the *single pontoon* (rectangular prism) or barge-type hull; the catamaran or multipontoon configuration with the deck spanning transversely between pontoon units; and the *semisubmersible* type, frequently used in mobile offshore drilling units, with a deck superstructure supported on any number of vertical, usually cylindrical, buoyancy columns, which are themselves often connected below the waterline to continuous submerged pontoonlike buoyancy units.

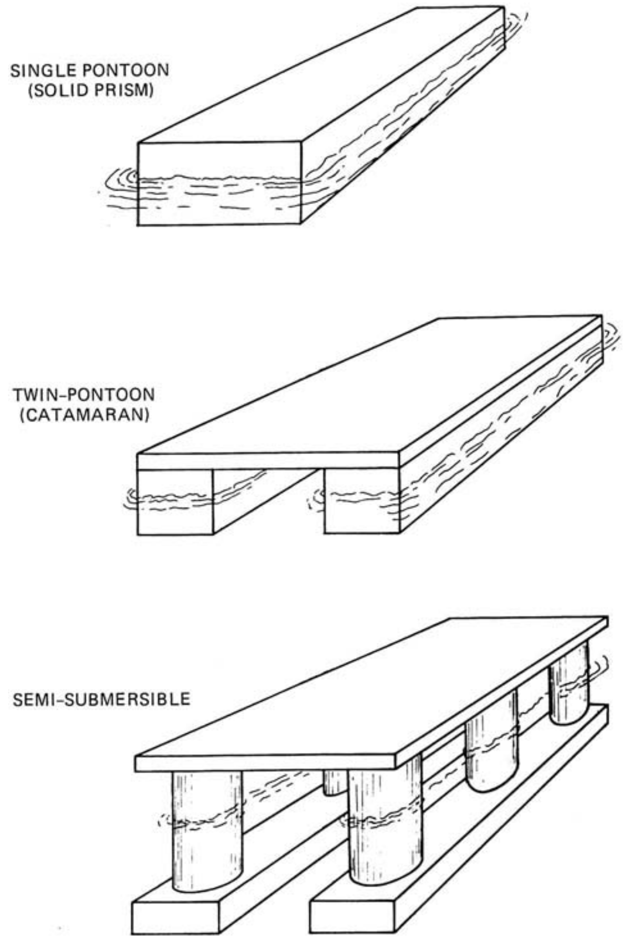


Fig. 9-1. Floating-pier basic hull types

The semisubmersible hull-type configuration has as its principal advantage that it is largely transparent to wave motion because of its minimal waterplane area. The natural period of roll, for example, of a semisubmersible unit usually is or can be made to be longer than that of typical ocean waves. In addition to its relatively small change in displacement during the passage of a wave, the platform remains relatively motionless compared to a barge-type hull of the same displacement. Mooring loads also are lower for this configuration compared to hulls of equal displacement. Its disadvantages include a relatively low deck-load capacity, sensitivity to moving weights on deck, and generally higher construction and operating costs. This structural configuration, however, would be advantageous at locations exposed to ocean waves where deck loadings are light, such as for an offshore tanker berth. The catamaran or multihull configuration may offer some cost savings or construction advantages over the single pontoon where an equal deck area is important but load capacity is not.

As it may not be feasible to construct a floating pier in one single length, individual pontoon units may be interconnected end to end by rigid hinges or semiflexible connections, or they may be joined together on-site in the water by posttensioning or welding to form a single unit that cannot be constructed or towed from its construction site in one piece. Modular pontoon float systems are commercially available and may be suitable and adapted to a wide range of uses, such as ferry and tour boat docks and other service craft applications. In some cases, existing barges may be modified for floating dock/port facility use. Alternative pontoon arrangements include individual pontoons spanned by a continuous deck system like a floating bridge, or a hinged deck system spanning between pontoon units; the latter is applicable only at sites with little or no wave action.

Floating piers may offer several advantages over conventional fixed-pier construction, depending upon local site conditions. They are most advantageous for these site conditions: very deep water, large tidal or river-stage variations, difficult foundation and/or seismic conditions, remote locations lacking construction-material availability or equipment access, and river or delta locations subject to migrating shorelines that may require the pier to be relocated. Other conditions favoring floating piers include temporary installations and those where speed of construction or rapid deployment is of prime importance, such as for military applications and environmentally sensitive sites where a floating structure may create less disturbance to water circulation, bottom habitat, and so on. Their principal disadvantages include higher maintenance and operating costs and susceptibility to wave action, resulting in downtime caused by unacceptable motions or to damage from storm waves. Other possible disadvantages include limited deck space and capacity and, sometimes, the requirement for cumbersome shore access or extensive mooring structures.

In many instances, floating piers may make up an entire marine terminal or port. One example is the Falkland Islands "Flexiport," shown under construction in Fig. 9-2, which consists of seven barges forming a 1,000-ft-long floating quay with transit sheds that can accommodate roll-on/roll-off (Ro/Ro) and general cargo. Shore connection is via a movable causeway on grounded pontoons (DHA 1983). The Port of Valdez, Alaska, has a container terminal consisting of a 700- by 100-ft prestressed concrete pontoon with a 40-ton container crane and two shore access bridges. Extreme waves for design approach 8 ft high, and the average tide range is 22 ft (PCI 1982, Zinserling and Cichanski 1982). The Port of Valdez, Alaska, oil dock berth No. 1 loading dock consists of a semisubmersible-type tubular steel space frame supporting a 390-by-70-ft deck buoyed by 22-ft-diameter vertical cylinders (Tsinker 1986). The Port of Iquitos, Peru, was upgraded in 1977 with a 614-ft by 50.5-ft wharf consisting of five rectangular steel pontoons added to an existing 285-ft by 30.5-ft pontoon that had operated with some intermittent upgrading for the previous 45 years. The wharf services general cargo vessels of up to 6,000 gross registered tons

Fig. 9-2. Falkland Islands "Flexiport" under construction. Access bridge is supported on removable submerged pontoons

Source: Photo courtesy of MacGregor-Navire

(GRT), and the river level varies by 35 ft (Tanner et al. 1983). Ahn et al. (2013) describe the design, construction, and testing of a planned floating container port.

Floating structures have been used as offshore terminals for the storage and transfer of hazardous cargoes such as the liquefied petroleum gas (LPG) storage facility in the Java Sea (Anderson 1973) (see Fig. 1-10) and a proposed liquefied natural gas (LNG) terminal (Anspach et al. 1980). The U.S. Navy has carried out detailed studies for a two-level-deck, modular floating pier to service naval vessels (Jahren and Davis 1986, Wernli and Springston 2008, Zueck and Wernli 2010). In some instances, great savings can be realized by recycling existing barges for use as piers. An automobile unloading facility was built in the Port of Wilmington, Delaware, using existing 250- by 35-ft car floats to form a 515-ft-long wharf with two floating access ramp barges (Felsburg 1977). Additional floating port case studies are provided by Tsinker (1986), who also provides an overall in-depth treatment of floating-pier design and construction. Floating pontoons provide a convenient means of transferring passengers at ferry terminals with large tide ranges, thus avoiding the need for adjustable ramps and boarding platforms. Keiser et al. (2013) describe the unique application of a pontoon for passenger transfer at a cruise ship terminal.

Very large floating structures (VLFSs) have been proposed for various applications, including marine terminals. Such structures would likely be based on semi-submersible-type hulls of integrated modular construction and subject to different behavior in waves because of the low resonant frequencies of the deformable hull. Che et al. (1992) analyze such a structure of 500-m length and determine hydrodynamic response coefficients and hull mode shapes. Another proposed alternative to reducing the deflections under load of very long but shallow-depth floating structures is the use of free-flooding "gill cells," as described by Pham and Wang (2010). More rigorous treatment of VLFS design can be found in Wang and Tay (2007) and of large concrete floating structures in ACI (1988, 2010). The design of much smaller floating docks for small craft is addressed in Section 9.8.

Link-Spans and Transfer Bridges

Transfer bridges or link-spans are required for the transfer of vehicular ferry and Ro/Ro traffic from ship to shore. Although some larger oceangoing vessels are equipped with their own ramps that span directly from the ship to the quay or wharf deck, most ports that handle Ro/Ro cargo, in particular, those with large water-level fluctuations, have link-span arrangements, which often are supported on floating structures. Floating link-spans may be arranged for end-on, side-on, or quarter-point loading operations. Floating link-spans can be readily moved and adjusted to suit tidal variations and vessels' threshold heights. Pontoons usually are fitted with pumping and flooding systems for ballast control, and some installations may have adjusting towers and mechanical compensating devices on their mooring systems. Many link-spans are moored directly alongside a pier or quay, as shown in Figs. 9-3 and 9-4. Link-span pontoons must be of ample size to minimize changes in draft and trim during cargo transfer. Normally, pontoons should not change trim more than approximately 1° during routine cargo transfer operations.

Camels

Camels are floating separators placed between a vessel and quay or between two vessels in order to maintain a safe standoff distance. A camel may consist simply of a timber log or group of logs lashed together, or of heavy timbers with struts and braces secured together in crib-work fashion. Floating fender units (see Section 5.5) also may serve as camels. Larger camels may be constructed of steel or timber frameworks with buoyancy tanks for flotation, or of steel pontoons. Sometimes, small barges are adapted for use as camels. The width of a camel is determined by the ship's roll characteristics, freeboard, and presence of overhangs, such as the flight decks of aircraft carriers. The minimum length of a camel usually is on the order of 30 ft or as required to maintain acceptable contact pressures with the vessel's hull or to distribute the load against a sufficient number of pier fender piles. Consideration should be given to the stability of the camel against lifting and overturning. Larger camels have open deck surfaces or walkways, for which a minimum deck load of 20 lb/ft^2 or more should be considered. Where steel pontoons or buoyancy tanks are used, they should have internal watertight compartments or be foam filled. Special deep-draft camels are required for berthing submarines. The U.S. Navy (DOD 2005a) has established certain design criteria for camels.

Fig. 9-3. Dockside movable "link-span" pontoon mounted transfer bridge for Ro/Ro cargo transfer at Melbourne, Australia

Source: Photo courtesy of MacGregor-Navire

Containment Booms

Containment booms are a common feature in most ports, especially at oil tanker berths. Booms must be readily deployable, and leave-in-place booms must be easy to move and resecure during vessel movements. Oil containment booms typically consist of the following basic elements: flotation units, which should have ample reserve capacity; a skirt; a continuous tension member located at or near the waterline; ballast to hold the skirt down and maintain stability; and anchorage points, usually not spaced more than 100 ft apart. Most booms today have synthetic

Fig. 9-4. Submersible-type movable link-span alongside quay at Douglas, Isle of Man

Source: Photo courtesy of MacGregor-Navire

fabric skirts coated with a hydrocarbon-resistant polymer, usually polyurethane, with encapsulated buoyancy elements near their tops and continuous wire or other tension-member material incorporated in them. A typical in-harbor boom has a total depth of about 3 ft with approximately 1 ft or less of freeboard, although booms intended for rough water service may need to be much larger. Booms often are put into hoop tension between mooring points, which may be increased by currents, wind, and tidal variations. Therefore, a tension capacity on the order of 10,000 to 15,000 lb often is required (Hubbell et al. 1978). End and mooring connections must be strong but easy to secure in the water. Booms must be sufficiently ballasted and rigid enough to maintain their shape and stability under wind and current loads. An overview of containment boom design and use is given by Gundersen (1990). Analysis of boom behavior in currents and waves is addressed by Swift et al. (1992), Bruno and Van Dyck (1997), and Fang and Johnston (2001). Floating booms may also be installed as debris and/or ice barriers. The design of ice booms is covered by Foltyn and Tuthill (1996).

Port Security Barriers

Floating port security barriers are a relatively common sight in many ports with military and/or high-security facilities. These barriers may be of various forms, ranging from low-profile boom-type barriers to warn off errant small craft at lower security sites to buoyant pontoons supporting large above-water barriers designed for high-speed boat impacts and sometimes including below-water barriers as well in high-security/high-risk port facilities. A common configuration, as shown in Fig. 9-5, consists of transverse float units with a continuous above-water fence of steel or ballistic mesh capable of resisting the impact of a design boat at a specified speed. Design criteria for boat barriers are discussed by Seelig and Taylor (2003), and additional technical reports on specific boat barrier designs and installations have been prepared by the U.S. Navy Facilities Engineering Service Center (NFESC).

Fig. 9-5. Port security barrier

Source: Photo courtesy of Appledore Marine Engineering, LLC

Single-Point Moorings

At offshore deepwater locations, transient tankers may be moored to single-point mooring buoys (SPMs) and their cargo may be transferred between ship and shore through hoses and underwater pipelines, or by lightering. More permanently moored floating production, storage, and offloading structures (FPSOs) may also be moored to SPMs. There are several variations of SPMs, such as a single-anchor leg mooring (SALM), which consists of a single bottom-anchored vertical chain or a rigid strut riser attached to a large floating buoy or yoke arrangement, or a catenary anchor leg mooring (CALM), which consists of a buoy anchored by several mooring chains to independent bottom anchors. Fig. 9-6 shows a VLCC transferring its cargo at an SPM, SALM terminal. This installation is described in some detail by Gruy et al. (1979). The design of these specialized facilities is outside of the scope of this book. A more detailed overview and design criteria can be found in ABS (1996), Benham et al. (1977), USACE (1976), and Pedersen (1975). Vessel moorings in general are covered in Section 9.7.

Floating Breakwaters

Floating breakwaters (FBWs) have been used with varying degrees of success, primarily in reducing the heights of short-period waves, of generally less than 4 s, at small-craft facilities in exposed locations. The state of the art of FBW design has been covered by PIANC (1994), and practical design considerations are addressed by

Fig. 9-6. VLCC discharging crude oil at one of three SALM buoys at the Louisiana Offshore Oil Port (LOOP)

Source: Photo courtesy SOFEC, Inc.

Gaythwaite (1987, 1988). Headland (1995) presents a concise introduction to the analysis of FBW performance and mooring forces, and many important papers on the subject can be found in the FBW conference proceedings (Kowalski 1974, Adee and Richey 1981). Although FBW design is beyond the scope of this text, a few important principles that apply to floating structures in general are reviewed here.

A rigid body moored normal to a wave train reflects some amount of wave energy back seaward and dissipates a much smaller amount through friction and turbulence. The result is a smaller transmitted wave height on the leeward side of the structure, although the transmitted wave has the same period as the incident wave. The incident wave height (H_i), the reflected wave height (H_r), and the transmitted wave height (H_t) are related by the following equation:

$$H_i^2 = H_r^2 + H_t^2 \tag{9-1}$$

The ratio of the transmitted height to the incident height is called the transmission coefficient:

$$C_T = H_t / H_i \tag{9-2}$$

Fig. 9-7 shows the approximate variation of C_T with the ratio of the structure width (B) to incident wavelength (L) for a rectangular object moored in regular

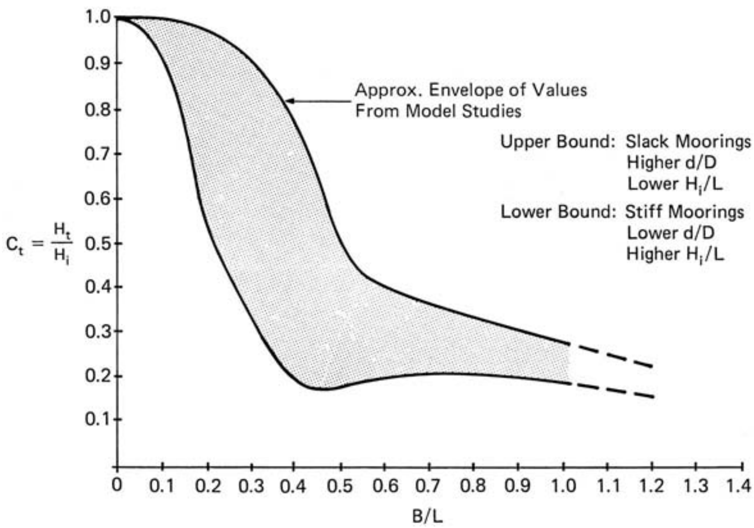


Fig. 9-7. Representative values of wave transmission coefficient (C_T) versus beam-to-wavelength ratio (B/L) for a floating rectangular prism

waves. Note that the actual value of C_T also varies with the relative water depth (d/D), the wave steepness (H/L), the mooring-system stiffness, and other factors. It is evident that when B/L is relatively large, say, on the order of 0.5 or greater, the FBW is an effective wave attenuator; and as B/L is reduced, more wave energy passes by the FBW. At a B/L of around 0.2 or less, the FBW essentially follows the wave contour with little or no wave attenuation. Reflected waves result in high mooring forces and cause standing wave patterns of increased wave height in front of the structure.

The mooring stiffness has an important effect on both wave attenuation and mooring loads (Isaacson and Fraser 1979). In general, the stiffer the mooring and the more wall-sided and deeper draft the structure, the greater the mooring loads and wave attenuation are. An important, often overlooked aspect of FBW effectiveness is its structural continuity along its length (Eranti and Alasiurua 1996) and the provision of a suitable overall length with regard to the incident wavelengths. In general, the FBW should be at least three wavelengths long to provide a minimum shadow zone and be structurally continuous over as long a length as feasible.

9.2 Hydrostatics and Stability

A floating object displaces a volume of water equal in weight to its own weight (the Archimedes principle). For example, the displacement for a rectangular object is given by the product of its length (L_s), beam (B), and draft (D) and the unit weight of the water, γ ($=64$ lb/ft^3 for seawater). For an irregularly shaped object such as a ship, the displacement can be found by multiplying $L_s \times B \times D \times C_B$, where C_B is a block coefficient used to account for the percentage of the volume of the ship's prism. Representative values of C_B and other form coefficients defining a vessel's shape were introduced in Section 2.2.

A floating object is in equilibrium when no external forces act on it, and its centers of gravity (c.g.) and buoyancy (c.b.) lie in the same vertical plane. If the body's equilibrium is disturbed by the application of a heeling moment (M_h) such as that induced by a moving weight on deck or a wind force, then the body heels, or lists, until a new equilibrium position is found, whereby a righting moment (M_r) formed by the object's new c.b. acting with its c.g. balances the heeling moment. The righting moment is given by

$$M_r = \Delta GM \sin\psi \tag{9-3}$$

where

Δ=object's displacement,
GM = its metacentric height, and
ψ = angle of heel.

Referring to Fig. 9-8a, GM is the vertical distance between the c.g. and the so-called metacenter (M), which is the point of intersection of the line of action of the object's buoyancy and the heeled centerline for small ψ. A righting lever arm, $GZ = GM \sin\psi$, forms a couple between the object's c.g. and c.b. GM can be found from

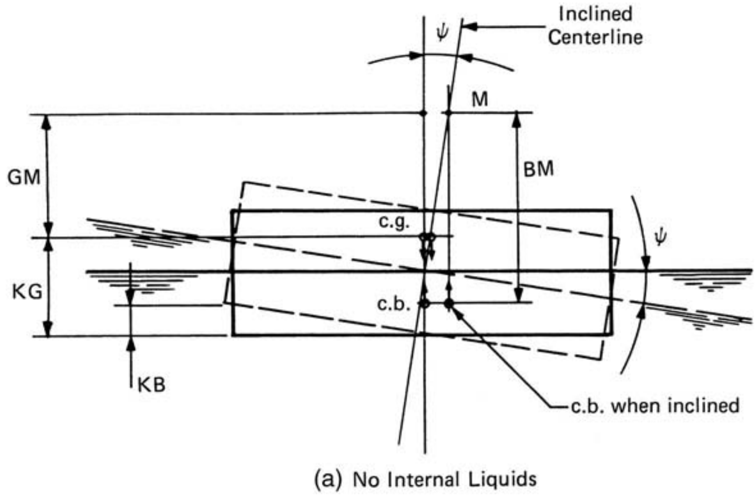
$$GM = BM + KB - KG \tag{9-4}$$

where

BM = vertical distance between c.b. and M and is equal to the moment of inertia of the waterplane area (I_{wp}) divided by the displaced volume (∇);
KB = distance from the object's bottom (or keel); and
KG = distance from the keel to the c.g.

In general, for waterplane areas of arbitrary shape, the value of I_{wp} must be found by integration.

The reader is referred to the basic naval architecture texts cited in Chapter 2 for a more comprehensive theoretical development of floating body stability. For the stability of linked and hinge-connected pontoons and catamaran hulls, refer to Tsinker (1986). It is important to note that if the pontoon has internal liquids (ballast water, leakage, and so on), then when it is heeled, the internal water sloshes (the "free-surface effect") and comes to a new equilibrium position with the c.g. of the internal water now shifted so as to increase the effective heeling moment (Fig. 9-8b). The effect of this internal free surface, which can be considered as a negative waterplane area, is to reduce the positive waterplane area. In this case, the



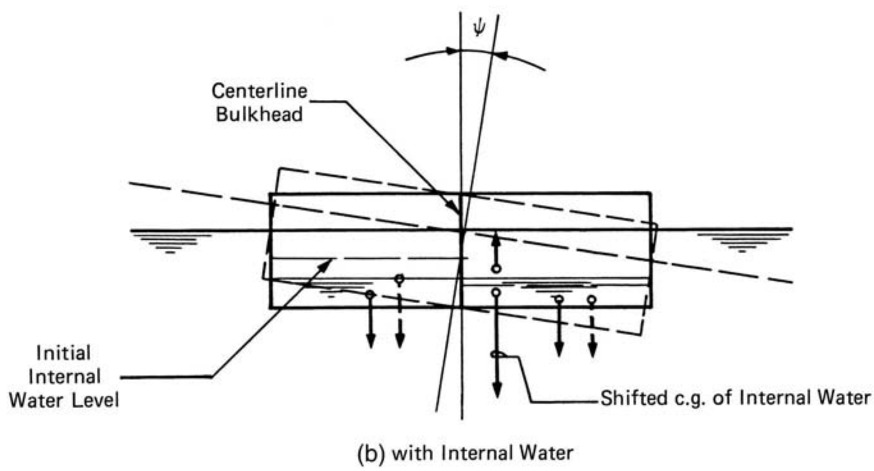


Fig. 9-8. Hydrostatic stability definition sketch: (a) Pontoon without internal water, (b) Pontoon with internal water

negative waterplane area (I_{neg}) is taken about the neutral axis of the internal waterplane area.

Although there are no definitive guidelines for minimum acceptable values of GM for floating piers, a minimum value on the order of 2 to 3 ft or perhaps 10% of the structure's beam under the worst anticipated load conditions seems a reasonable assumption. High values of GM are desirable for initial stability, but there may be some instances where low values are sought in order to increase the period of roll, which is inversely proportional to the square root of GM.

The moment required to list the pontoon a given amount, say 1 in., can be calculated from the GM as

$$\mathrm{MLI} = \Delta GM_T/12B \tag{9-5}$$

where
GM_T = transverse metacentric height, and
MLI=moment to list 1 in.

Similar calculations can be carried out about the pontoon's centerline axis or midship section to obtain the longitudinal GM_L and the moment to trim 1 in. (MTI):

$$\mathrm{MTI} = \Delta GM_L/12L_S \tag{9-6}$$

The side-to-side or end-to-end trim given by Eqs. (9-5) and (9-6) gives the total amount of trim or list, which is equal at each end for a rectangular shape and must be proportioned by the distance to the center of flotation for an irregular waterplane area.

It often is convenient to know how much the float sinks under a given load. The load required to sink the pontoon 1 in. is termed the pounds per inch immersion, PPI, as given by

$$\mathrm{PPI} = \mathrm{WPA} \times \gamma/12 \tag{9-7}$$

where
WPA=waterplane area in square feet, and
γ = unit weight of water in pounds per cubic foot.

The change in draft of the pontoon caused by an added weight anywhere on deck can be found by superimposing the change in draft caused by direct sinkage (PPI), heeling moment (MLI), and trimming moment (MTI). Maximum allowable angles of trim and list vary with applications but in general are not likely to exceed 1° or 2° with perhaps up to 5° to 6° (1:10 deck slope) allowed under extreme conditions. Minimum freeboard requirements may vary widely with vessels to be serviced, but a reasonable amount should be maintained for reserve buoyancy and to prevent wave overtopping.

The overall hydrostatic and stability properties versus draft of any floating structure can be plotted in graphical form, as illustrated in Fig. 10-34 for a floating dry dock. Floating piers and similar structures must be subdivided internally by watertight bulkheads to provide adequate stability with internal water and under damage conditions. Damage stability criteria should specify minimum required freeboards and trim/list with one or more compartments flooded. Floating-pier intact freeboard requirements vary but generally range from 4 to 6 ft or more for the berthing of larger coastal and oceangoing vessels.

9.3 Motion Response

The general problem of analysis of a moored floating body, which is most complex, was introduced in Chapter 6. This section considers basic concepts that are of practical application in the design of floating piers. Considering the six degrees of freedom of movement of an unrestrained or partially restrained floating body, described in Section 6.1 and illustrated in Fig. 6-2, three motions—heave, roll, and pitch—have gravity as a restoring force and thus exhibit natural periods of free oscillation. Of these three modes, heave and roll (i.e., vertical rise and fall and side-to-side rotation) are of particular significance in floating-pier design. Pitching motion (end-for-end rotation) usually is not important because of the relatively long length of a floating-pier structure compared to the length of the incident waves. Exceptions would be locations exposed to long-period swell propagating along the longitudinal axis of the pier, or piers such as landing floats with low aspect ratios of length to beam (L_s/B). In the latter case, a clear distinction between pitch and roll cannot be made. The remaining three modes of motion—surge, sway, and yaw—may be significant in piers moored with slack chains or cables, but they are seldom of critical importance in rigidly moored floating-pier design.

The heave response normally is measured in terms of the ratio of heave amplitude to incident wave amplitude, the roll response as the structure-slope-to-wave-slope ratio, and sway motion as the ratio of sway amplitude to wave amplitude. The trajectory of the center of gravity of the structure under wave action varies with the wavelength and wave height in relation to the water depth. In deep water and with steep waves, the motion ellipse is almost circular, with heave and sway amplitudes almost equal. In shallow water and for low-amplitude waves, the wave particle orbits become elliptical, and heave motions are reduced, but sway amplitudes remain relatively large. The relative motion response of a floating structure also depends importantly upon the relative water-depth-to-draft ratio (d/D), the structure-beam-to-draft ratio (B/D) as it affects the virtual mass of the structure, and the beam-to-wavelength ratio (B/L), as well as the wave direction and the degree of mooring restraint.

Acceptable limits to motion usually are dictated by the human response, although certain cargo-handling equipment and operations may have more severe limits. Rosen and Kit (1985) arrived at the following limits in their study of acceptable motions within small-craft harbors:

- Maximum linear accelerations: 1.3 ft^2/s
- Maximum angular accelerations: 2° per s^2
- Maximum peak roll-to-roll motion: 6°

Chapter 6 presented some general information on the acceptable motions of berthed ships versus type of operations. Faltinsen (1990) presents a summary of threshold motions for ships and offshore structures.

Because lateral sway motions usually are sufficiently restrained by the pier's moorings, the remaining discussion concerns heave and roll motions, both of which can be greatly amplified when the natural period is near that of incident waves of sufficient height and length to excite resonant motions. Fortunately, both heave and roll can be controlled to some degree by varying certain design parameters. Both heave and roll are affected by the amount of damping present, but only heaving is significantly affected by the added mass effect (introduced in Section 5.3).

Rolling may be affected by mooring restraint, which increases damping and thus reduces maximum amplitudes but does not significantly alter the natural period. For the general case of unrestrained rolling, roll motion characteristics can be solved from the following equation:

$$I_M \frac{\partial^2 \psi}{\partial t^2} + C_d \frac{\partial \psi}{\partial t} + M_r = 0 \tag{9-8}$$

where

I_M = structure's mass moment of inertia, including the virtual mass of water about its longitudinal axis;

ψ = roll angle;

C_d = damping coefficient; and

M_r = righting moment=$\Delta GM \sin\psi$ [Eq. (9-3)].

In general, the added mass and damping can be neglected for unresisted rolling in calm water, especially at small values of ψ. This is not true in general for pitching motions, where the added mass in particular may have a significant effect in increasing the natural pitch period. The general solution to Eq. (9-8), neglecting the damping term, is

$$\psi = \psi_0 \cos \omega_n t + \frac{V_0}{\omega} \sin \omega_n t \tag{9-9}$$

which gives the value of ψ as a function of time (t), with ψ_0 and V_0 the initial angle and angular velocity. The quantity ω_n is the natural circular frequency, and ω is the wave circular frequency ($2\pi/T$), where it can be shown that

$$\omega_n^2 = \frac{\Delta GM_T}{I_M} \tag{9-10}$$

Here, ΔGM_T is the spring constant, I_M is equal to $\Delta K_r^2/g$, where K_r is the radius of gyration, and ω_n equals $2\pi/T_n$ by definition. By substituting into Eq. (9-10) and rearranging to solve for T_r, the natural period of roll is found to be

$$\omega_n^2 = \frac{\Delta GM_T}{I_M} \tag{9-11}$$

which in feet-per-second can be further reduced to

$$T_r = 2\pi \sqrt{\frac{K_r^2}{gGM_T}} \tag{9-12}$$

For most ships, K_r is usually in the range of $0.36B$ to $0.45B$, and for a rectangular pontoon, K_r is generally in the range of $0.29B$ to $0.35B$ and varies with beam-to-hull depth ratio (B/D_s) decreasing with increasing B/D_s. Offshore barges typically have a B/D_s in the range of 2.0 to 3.0, and most pontoon applications of interest herein are generally in the range of 2.5 to 4.0. Therefore, the natural period in roll for a rectangular pontoon ($B/D_s = 3.0$) can be estimated from

$$T_r = \frac{0.37}{\sqrt{GM_T}} \tag{9-13}$$

The angular acceleration in roll (α_r) affects personnel and equipment operations and should be kept as low as possible. For structures designed to be towed to remote sites, which may be subjected to much more severe wave conditions than would be expected during operations, the acceleration in roll may govern the design of appurtenances. It can be shown that

$$\alpha_r = \frac{4\pi^2}{T_n^2}\psi \tag{9-14}$$

where ψ is the peak roll angle in radians. The tangential force (F_t) acting on an object of weight W at a distance r from the roll axis is

$$F_t = \frac{W}{g}\frac{4\pi^2}{T_n^2} r\psi \tag{9-15}$$

Heave motions can be expressed in the form of a second-order differential equation similar to Eq. (9-8). Damping can be safely neglected for spud guide-type and most catenary chain-type moorings, but added mass effects cannot. The natural heave frequency (ω_h) can be solved in similar fashion to Eq. (9-10), where the spring constant is equal to the product of the waterplane area and the unit weight of water, and the mass is taken as the structure's displacement multiplied by an added mass coefficient (C_m) (see Section 5.3). Hence, the natural period in heave (T_n) for a rectangular pontoon is found from

$$\begin{aligned} T_h &= 2\pi\sqrt{\frac{\gamma C_m(L_s BD)}{\gamma g(L_s B)}} \\ &= 1.108\sqrt{C_m D} \end{aligned} \tag{9-16}$$

The value of C_m may vary greatly with pontoon properties (B/D) and relative water depth (d/D). Fig. 9-9 shows T_h versus draft (D) for a range of B/D ratios for conditions with $d/D \geq 5$. Theoretical values of C_m were calculated on the basis of information provided in Blevins (1979). The effect of increasingly shallow water is to increase C_m and hence also T_h.

As measured in terms of the heave-amplitude-to-wave-amplitude ratio, $a_h/a_w = 2a_h/H_i$, heave motion is highly dependent upon the structure-beam-to-wavelength ratio, B/L. There is, then, a frequency-dependent response that can be

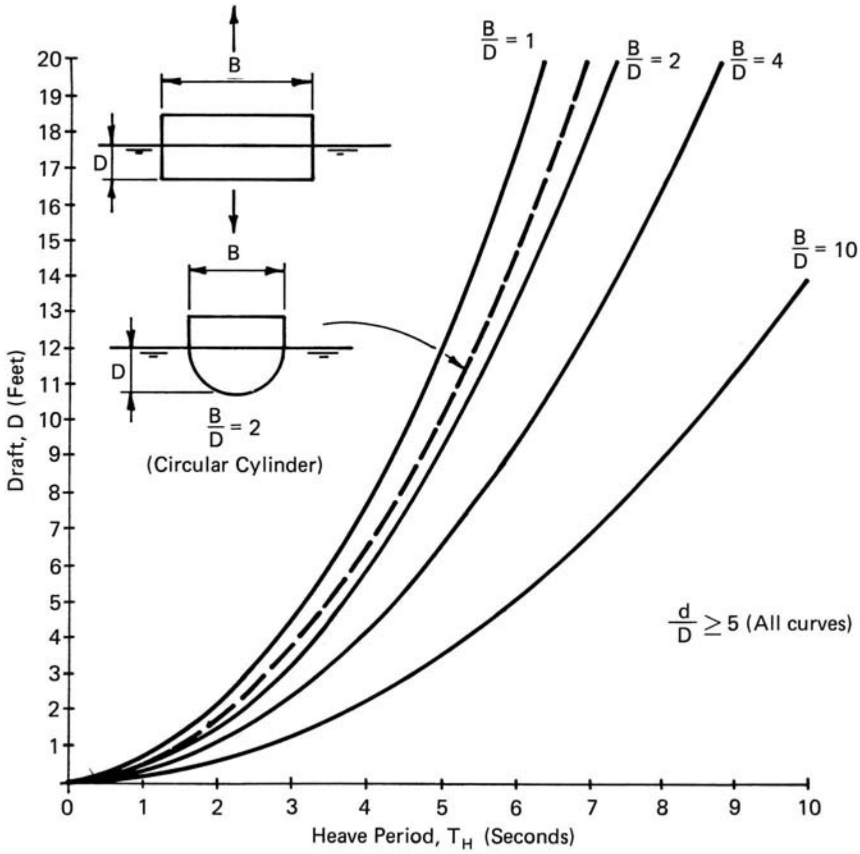


Fig. 9-9. Natural period of heave versus draft for rectangular pontoons

subjected to a spectral analysis, as introduced in Section 6.6, to determine the structure's response in irregular seas. Fig. 9-10 illustrates the heave response spectrum for a 30-ft-wide pontoon ballasted to a 7.5-ft draft and subjected to an incident wave spectrum with a spectral peak period, T_p, of 3.9 s and a significant wave height, H_s, of 3.0 ft. The response amplitude operator (RAO) or transfer function curve is based upon theory for a slack-moored rectangular object in a beam-sea condition with a value of 4.0 for B/D (Isaacson and Fraser 1979). The peak in the RAO near T_h is not so pronounced in actual model tests or in full-scale observations. In general, the heave response is negligible for $L < 0.75B$ and near unity for $L > 4B$. Note that this is a one-dimensional spectrum and does not account for the short-crestedness of real waves. Therefore, if the structure length, L_s, is much longer than the incident wavelength, the overall heave response is greatly diminished. Applications of directional spectra accounting for wave direction and short-crestedness are beyond the scope of this text. The reader is referred to the references cited in Chapter 6 for further information.

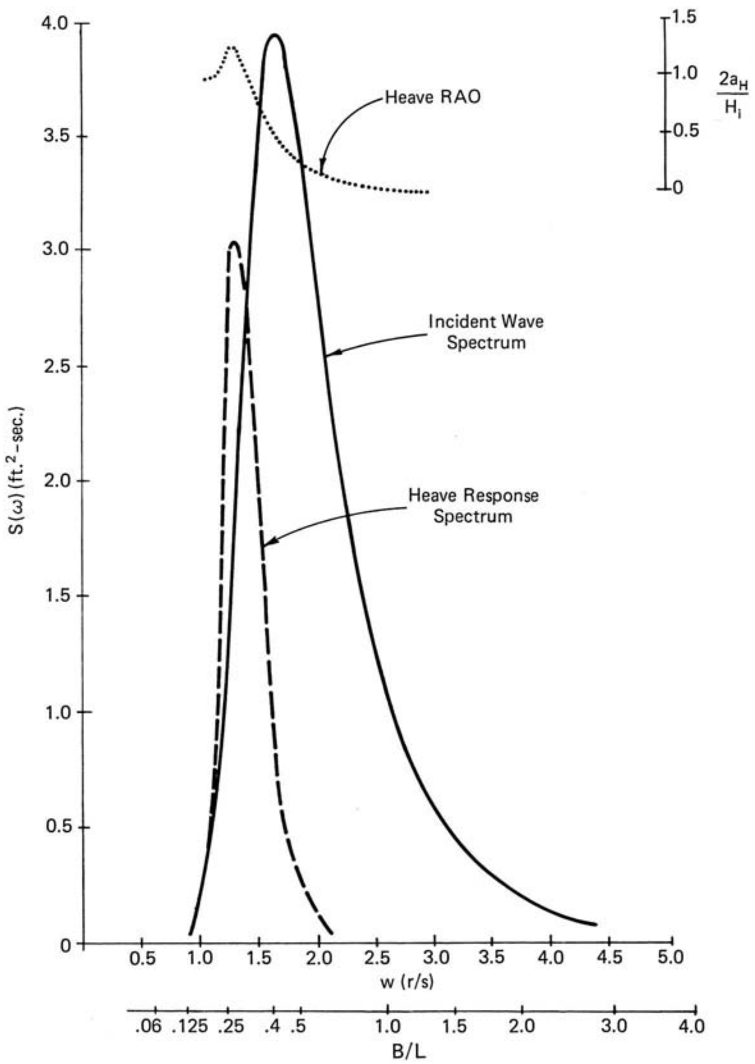


Fig. 9-10. Example heave-response spectrum for 30-ft-wide pontoon

Although waves are the usual source of excitation, it is important to consider the speed of traffic or traveling equipment that could initiate resonant motions. This is especially true with regard to the heave response of relatively small pontoons supporting link-span transfer bridges or floating-bridge spans.

The maximum vertical acceleration (α_h) in heave is given by

$$\alpha_h = \frac{4\pi^2}{T^2} a_h \tag{9-17}$$

Vertical accelerations are seldom a problem structurally but may be critical in determining separation forces and relative motions between access bridges and mooring compensation devices. Prolonged exposure to accelerations as low as approximately $0.05g$ may affect personnel performance, especially if coupled with roll or pitch motions and depending upon the period of oscillation.

9.4 Structural Design of Pontoons

Floating structures such as piers and landing floats are designed to accommodate the same kinds of deck live loads, vehicular and equipment loads, and vessel berthing and mooring loads as their fixed-pier counterparts, with these main differences: reaction forces caused by deck loads always are resisted by a uniformly distributed buoyancy force over the structure's bottom, the structure is subject to floating body motions and hence dynamic effects not experienced by fixed structures, and horizontal forces are resisted at discrete mooring points and must be distributed through the hull structure. It is interesting to note that although floating structures themselves are isolated from seismic forces, large forces may be developed at their mooring points. This is especially true for massive structures moored to rigid dolphins or piers in areas where strong ground motions are possible.

In general, floating structures must consider the following load categories:

- Deck loads that induce still water shear forces and bending moments and torsion effects;
- Wave-induced shear, bending, and torsion;
- Local hydrostatic pressures, including dynamic effects of wave action and sloshing in tanks;
- Mooring loads caused by vessel berthing and mooring and direct environmental loads against the structure;
- Thermal loads; and
- Launching, dry docking, and towing loads.

Consideration also must be given to fatigue, impact, corrosion, and mechanical deterioration. Connection points between interconnected floating units and moorings tend to be the most problematical and deserve careful analysis and detailing. Contemporary floating piers are usually of steel or reinforced or prestressed concrete construction. Smaller floats (see Section 9.8) also may be constructed of timber, aluminum alloys, and various plastics such as fiberglass (FRP). There are as yet no definitive codes or standards that govern the design of floating-pier structures. However, some guidance is given in the rules of the vessel classification societies such as the American Bureau of Shipping (ABS 2015a, b) or their foreign counterparts for steel structures such as Det Norske Veritas (DNV) and NKK, in ACI (1988, 2010) and FIP (1986) for concrete structures, and for FPS design and construction, in API (2011). General guidance on design principles and structural design of steel floating structures may be obtained from basic naval architectural texts such as those cited in

Chapter 2. The floating-structure design conferences, such as those at the University of Wisconsin (UWI 1990–1992) contain helpful design guidance for a large variety of structure types. Basic concrete and steel building codes also may be used for structural design, provided that appropriate load factors and allowable stresses are applied, and due consideration is given to minimum thickness of members, continuity of welds, and other factors relating to possible fatigue and corrosion problems.

The overall hull strength of a floating pier is governed by its gross-section modulus as determined from maximum still water bending moments caused by deck and equipment loads and/or wave loading. Wave loadings govern the design of ships and are also likely to govern the design of floating structures subject to ocean towing or at sites where wave heights may exceed 4 to 6 ft, depending on the ratio of structure length to wavelength (L_s/L). After being initially proportioned for the maximum longitudinal bending moment, the hull girder is checked for torsion and local combined load effects and strengthened as required. Because the framing of a rectangular pontoon typically is uniform throughout, the unloaded structure is uniformly supported over its length so that there is no still water bending moment associated with its dead load. Deck superstructures or moving equipment loads usually can be resolved into point loads in calculating the overall shear, bending, and torsion moments, the reaction to which is a uniform buoyancy force distributed along the pontoon bottom. Wave-induced bending moments are found by integrating the shear force (V_x) along the length of the pontoon:

$$M_x = \int_0^x V_x dx \tag{9-18}$$

where the shear force is obtained by integrating the net buoyancy force ($b - w$) along the structure length, with b the buoyancy force and w the weight per unit length of structure. Note that local concentrated loads can be deducted from the elemental buoyancy force to consider both still water effects and wave effects simultaneously. The shear force, therefore, is found from

$$V_x = \int_0^{L_s} (b_x - w_x) dx \tag{9-19}$$

Assuming a sinusoidal wave form, b_x is given by the surface elevation (z) times γB, with z varying along the wavelength as

$$z = \frac{H}{2} \cos \frac{2\pi x}{L}$$

The integration is carried out for the case of $L_s/L = 1.0$ with the wave crest at the structure midships (Fig. 9-11). This is called the "hogging" condition, as opposed to the "sagging" condition, in which the wave crests are at the structure ends. Sagging moments are found to be of equal magnitude but opposite sign to hogging moments. The shear force is then

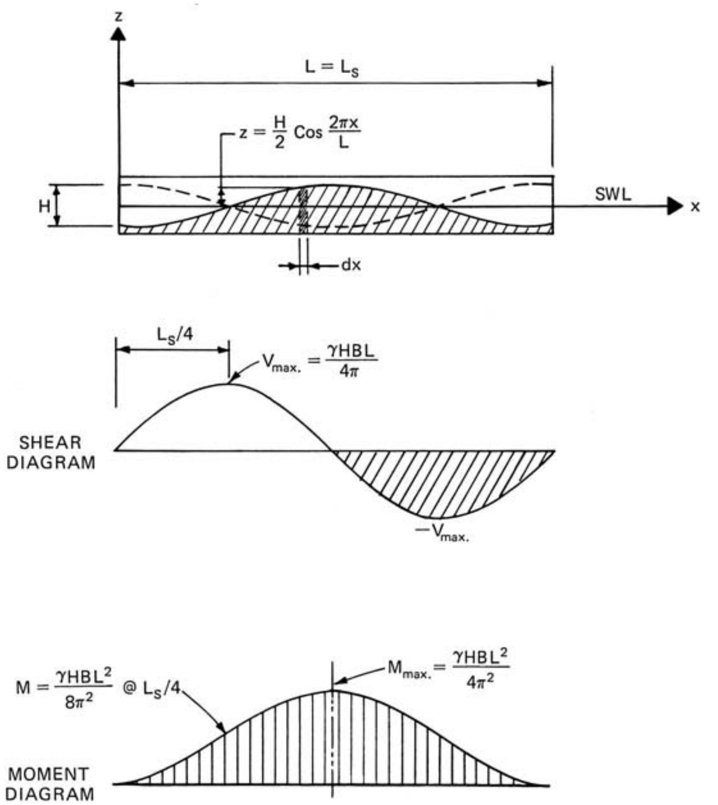


Fig. 9-11. Shear and bending moment for a rectangular structure in waves with wavelength equal to structure length

$$
\begin{aligned}
V &= \gamma B \frac{H}{2} \int \cos \frac{2\pi x}{L} \\
&= \gamma B \frac{H}{2} \times \frac{L}{2\pi} \sin \frac{2\pi x}{L} + C_1
\end{aligned}
$$

where the constant of integration $C_1 = 0$, given that $V = 0$, at $x = L_s/2$. The maximum shear force at the quarter points, $L_s/4$, is then

$$V_0 = \frac{\gamma HBL}{4\pi} \tag{9-20}$$

The bending moment is found from

$$
\begin{aligned}
M &= V_0 \int \sin \frac{2\pi x}{L} \, dx \\
&= -\frac{V_0 L}{2\pi} \cos \frac{2\pi x}{L} + C_2
\end{aligned} \tag{9-21}
$$

For the boundary conditions,

$$M = 0 \text{ at } x = 0, C_2 = \frac{V_0 L}{2\pi} \tag{9-22}$$

and

$$M_0 = \frac{V_0 L}{2\pi}\left(1 - \cos\frac{2\pi x}{L}\right) = \frac{V_0 L}{\pi} \text{ at } x = L/2 \quad M_0 = \frac{\gamma HBL^2}{4\pi^2} \tag{9-23}$$

The quantities M_0 and V_0 are known as the characteristic shear and moment, respectively, for $L_s/L = 1.0$. Worked example problems have been provided by Tsinker (1986). If similar calculations are carried out over a range of L_s/L, the maximum moment is found to occur at midships for $L_s/L = 1.12$ and is equal to 1.028 M_0. The midship bending moment is theoretically zero for $L_s/L = 2.0$ and is generally small for $L_s/L > 2.0$ and for $L_s \ll L$. Fig. 9-12, adapted from Muller (Gerwick 1975), shows the variation in moment with L_s/L. The maximum shear and moment in foot-pound units then are given by

$$V_{\max} = 5.24\, HBL_s \tag{9-24}$$

$$M_{\max} = 1.67\, HBL_s^2 \tag{9-25}$$

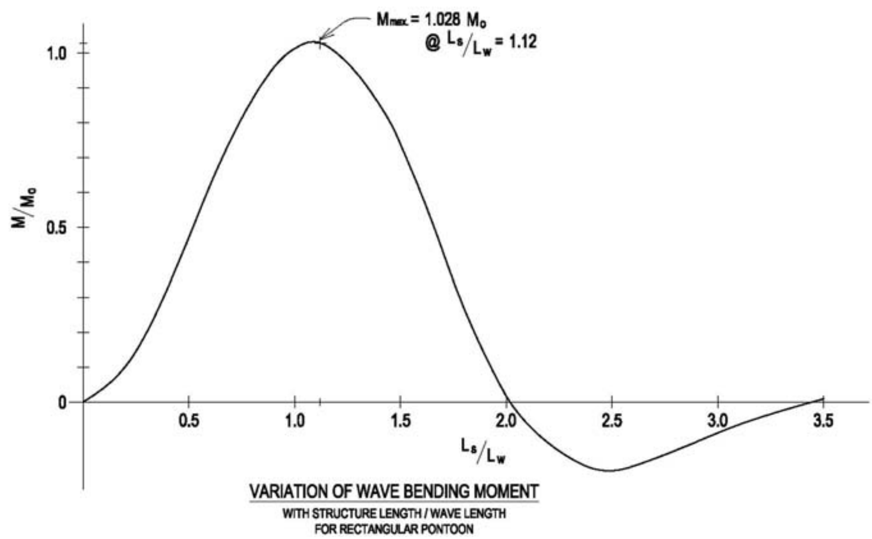


Fig. 9-12. Variation of longitudinal bending moment to characteristic moment (M/M_o) with pontoon length to wave length ratio (L_s/L_w)

Source: Gerwick (1975)

Torsional stresses may be manifested as wracking of sections in a primarily vertical or horizontal mode, warping out of planeness, and overall twisting action. Maximum torsional moments usually are calculated on the basis of the structure being supported along its opposing quarter points by wave crests. The angle of incidence of the waves and their length required to produce the maximum torque varies with the structure-length-to-beam ratio (L_s/B). A more rigorous discussion has been provided by Muller in Gerwick (1975). According to Muller, the maximum torque moment in foot-pound units for a rectangular shape is

$$M_T = 2.53\, HB^2 L_s \tag{9-26}$$

The maximum torque moment usually is found to be on the order of 20% of the longitudinal bending moment. Torsional stresses that are additive to direct and shear stresses associated with longitudinal bending are seldom a problem for the closed-box sections of steel hulls. For concrete, especially nonprestressed concrete, combined shear stresses may be a problem. The following combinations of shear stresses should be checked (Gerwick 1975):

1. $V_{\max} + 1/4 M_{T\max}$
2. $M_{T\max} + 1/4 V_{\max}$
3. $3/4\ (V_{\max} + M_{T\max})$

Hull deflections (δ) caused by bending are found by double integration of the moment-curvature equation and may be expressed semiempirically by an equation of the form

$$\delta = C_\delta \frac{M L_s^2}{EI} \tag{9-27}$$

where E and I are the modulus of elasticity and moment of inertia of the hull girder, respectively. The coefficient C_δ is often taken as 0.09 for ship hull shapes in waves (Taggart 1980). Hull deflections caused by operational loads are calculated using weight versus buoyancy distribution and elastic beam principles. Pontoons usually have a depth (D_s) greater than approximately $L_s/20$ in order to practically meet strength and deflection criteria. For vessels, L_s/D_s must not exceed 15 (ABS 2015a). For inland barges and floating piers, L_s/D_s generally should be less than 18 to 20.

Floating piers may be framed similarly to vessels either transversely or longitudinally (see Section 2.3), although transverse framing of open truss or rigid frames is most common (Fig. 9-13). Concrete hulls usually are modular rigid frames or Vierendeel trusses with vertical sides and bulkheads haunched where they meet the deck and bottom. In either case, the deck and bottom form the girder flanges, and the sides and longitudinal bulkheads form the webs in resisting overall shear and

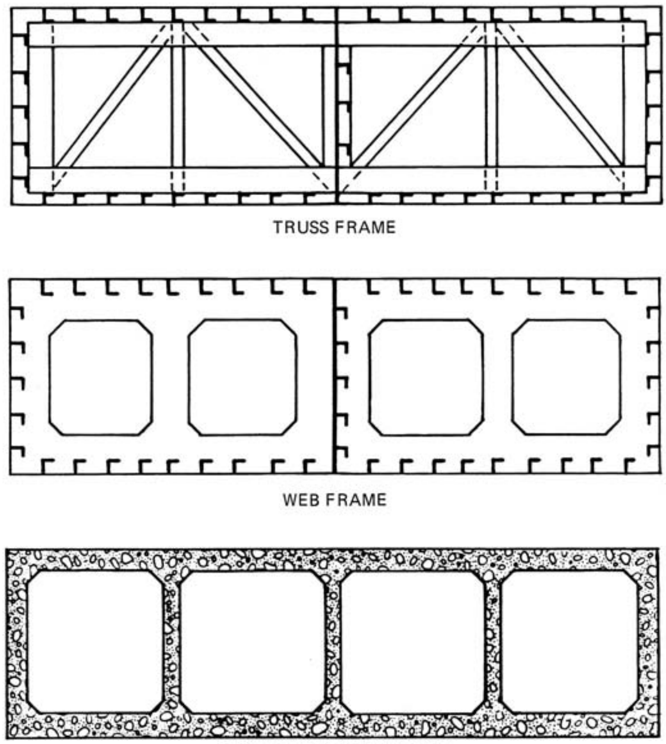


Fig. 9-13. Typical pontoon-framing schemes

bending. The frames must resist overall transverse bending moments, which are usually relatively small except in cases where heavy concentrated loads act along the float centerline, as in the case of floating dry docks (see Section 10.4). The frames also act to distribute vertical buoyancy forces and horizontal hydrostatic pressures.

Internal watertight bulkheads also must resist hydrostatic pressures caused by the differential head conditions between compartments. In addition to head pressures associated with the pier's maximum operating draft, a nominal equivalent head pressure acting above the deck level may be specified to account for the effects of overwashing waves or the impact contingency on the bottom and side shells. Japanese harbor standards (OCADI 2008) require that floating-pier bottom and sides be designed for a 0.5-m (1.6-ft) head above the freeboard deck level. Allowable stresses may be increased on the order of 75% to 90% of yield stress in steel structures under extreme load pressures. Bottom and side shells are always in a biaxial or triaxial state of stress and must be adequately proportioned to meet primary, secondary, and tertiary stresses associated with the overall hull bending, the local secondary bending between bulkheads or frame elements, and the local tertiary

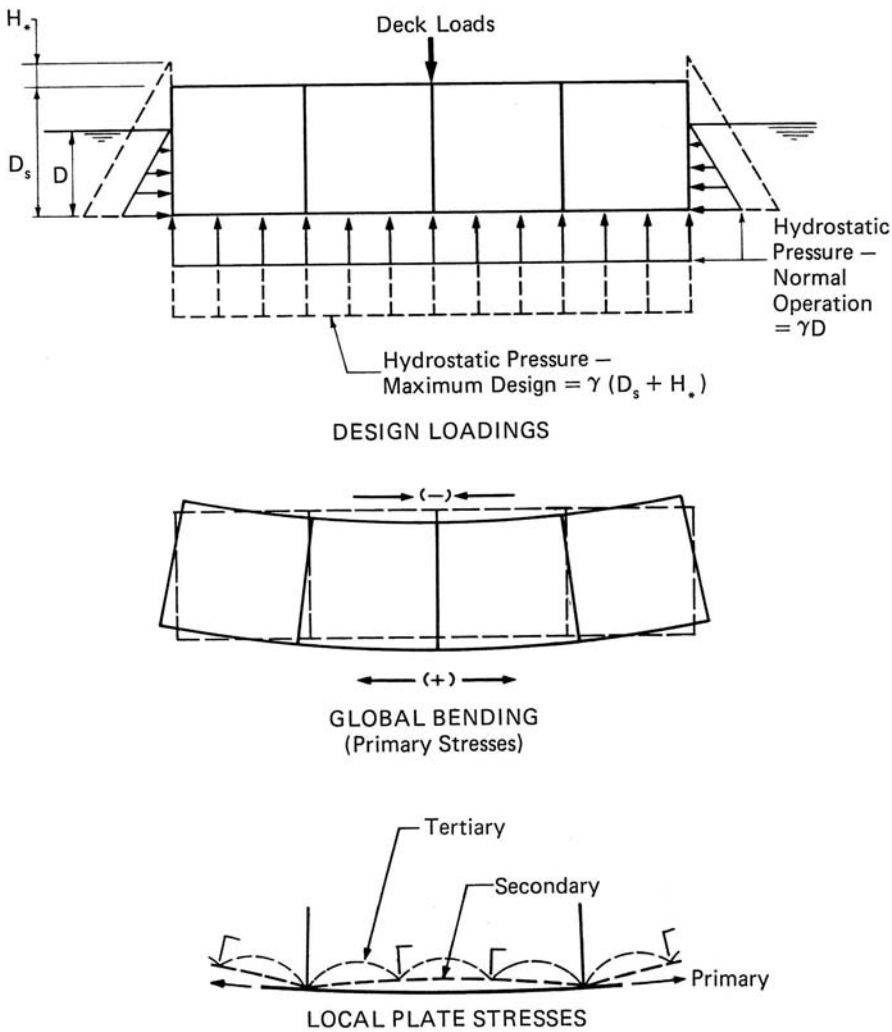


Fig. 9-14. Schematic of global and local hull stresses

bending between shell plating stiffening elements in steel structures (Fig. 9-14). Floating piers installed at freezing locations may need to be strengthened for ice forces. A general discussion of ice forces is provided in Section 4.5. For floating structures beset in solid ice, Noble (1983) and Liljestrom (1983) provide useful information.

Deck design is usually governed by local wheel load or crane float requirements. Although floating piers usually are not used for the storage of cargoes, locally high, uniformly distributed live loads may have to be considered for the temporary placement of heavy palletized or containerized loads. Orthotropic bridge-deck-type construction with either open- or closed-rib stiffening elements may be used in steel hull construction. Highway bridge design codes (see Section 4.2) are useful in determining wheel load distribution and deck framing requirements. Steel decks often are covered with cement concrete, bituminous concrete, or timber decking, the purpose of which is to provide a durable, skid-resistant, and easily maintainable wearing surface. Deck coverings should be kept as lightweight as possible. Tsinker (1986) provides a more comprehensive discussion of deck coverings and details.

Allowable stresses and load combination factors for steel structures can be determined with guidance from ABS (2015b) and ACI (1988, 2010), which provide guidance for design of concrete floating structures. Lightweight concrete with fresh unit weights of 120 to 125 lb/ft^2 can be reliably produced with compressive strengths in excess of 6,500 lb/in.^{2} (ACI 1988). As with fixed concrete structures, concrete mixes should be sulfate-resistant and proportioned for low permeability. The amount of reinforcing steel cover is critical to the typically thin sections of concrete floating structures and can be as low as 3/8 to 1/2 in. (ACI 1988). The necessary cover depends upon crack-control requirements (maximum crack widths of 0.01 in. should be considered for watertight elements), permeability, and the likelihood of degradation during normal service. Structures exposed to significant wave action should be designed to fatigue-stress criteria. For a 20-year design life, it usually is considered that the structure will be subjected to 10^8 wave cycles, corresponding to an average wave period of 6.3 s. Other temporary design conditions include launching and towing stresses, for which higher allowable stresses normally are justified. Before the commissioning of a new structure, internal tanks or compartments should be tested for watertight integrity by hose test or by air pressurization.

9.5 Ballast Control and Ancillary Features

Many types of floating structures of interest herein may be equipped with pumping and flooding systems for ballast control, including tank interior ventilation and sounding and personnel access features. Floating structures also may be outfitted with mooring hardware, fender systems, crane trackage, handrails and curbs, cargo handling and ship services, and lighting and fire protection systems similar to those of fixed piers (see Sections 7.6 through 7.8). The design of pumping and flooding systems is beyond the scope of this text, but a few basic principles are presented below.

Pumping Systems

Floating dry docks, link-spans, and some piers that are fitted with ballast systems generally require pumps capable of removing large volumes of water as rapidly as possible under relatively low but variable head conditions. Dewatering pumps most ideally suited to this purpose are usually of the axial mixed-flow or propeller type. The total dynamic head (H_D) used for pump selection is the sum of the hydrostatic head (h) and the hydraulic system losses. The net positive suction head (NPSH) is defined as the effective pressure head that causes liquid to flow through the suction piping (if any) and enter the eye of the impeller. The NPSH required by a given pump is a function of the pump design and is determined by the pump manufacturer. The available NPSH must equal or exceed the required NPSH. When the liquid source is above the pump impeller, it is equal to the sum of the equivalent head of the barometric pressure, plus the static head of liquid above the suction, minus any friction losses in the piping, minus the liquid vapor pressure.

There is a minimum height of water above the suction bell, referred to as the minimum submergence, below which the pump begins to develop vortices and ceases to operate efficiently. The depth of water remaining in the pontoon after the pump's "break suction" may create a problem for efficient operation, and it precludes inspection of the bottom. Therefore, pumps may be installed in depressed sump pits, or smaller stripper pumps may be installed as a remedy. In an efficiently designed sump, the height of the suction bell above the bottom plate usually is about one-half the suction bell diameter, and the distance from the side plate about one-fourth the suction bell diameter, depending upon the proximity of the tank or sump sides (Dicmas 1967). Flow velocities in the pump channel and at the suction bell must be properly controlled. Baffle plates and "umbrellas" fitted at the suction bell may help reduce vortexing and reduce the depth of residual water. Pump discharges normally are fitted with one-way flap valves to prevent water from entering because of wave wash or during deep submergence.

Pumps usually are driven by electric motors. The horsepower required is given by

$$\mathrm{BHP} = \frac{\mathrm{GPM} \times H_D \times s.g.}{3960 \times Eff} \tag{9-28}$$

where
BHP=brake horsepower of the motor,
H_D = head in feet,
GPM=gallons per minute,
s.g.=specific gravity of the liquid, and
Eff=efficiency.

Pump manufacturers publish performance curves of head requirements and efficiency versus capacity to aid in the selection of pumps operating over a range of head conditions. The final selection of the optimum pumping system often represents a compromise among initial cost, the cost and availability of electrical power, and the desired pumping time. Pumping systems for basin dry docks and for floating dry docks have their own particular requirements, as discussed in Chapter 10.

Flooding Systems

Flooding systems, like dewatering systems, are designed to meet some maximum allowable flooding time criteria. Consideration also must be given to varying head conditions, permissible flow velocities, and the size and the number of flood valves because they affect cost and operation.

The flow rate through a flood valve opening is given by

$$Q = C_0 A_0 \sqrt{2gh} \tag{9-29}$$

where

Q = flooding rate, usually in cubic feet per second (ft^3/s);
C_0 = discharge coefficient, which depends upon the flow velocity and contraction at the orifice and is usually taken to be on the order of 0.6 for circular orifices;
A_0 = area of the orifice; and
h = head, in feet.

Maximum flow velocities generally should be kept below 25 ft/s. The time rate of the flow under a varying head condition is found from

$$Q/A_0 = \frac{dh}{dt} \tag{9-30}$$

Rearranging and substituting,

$$dt = \frac{A_{wp}}{C_0 A_0 \sqrt{2gh}} dh \tag{9-31}$$

where A_{wp} is the waterplane area of the tank, which is assumed to be constant. Then,

$$\Delta t = -\int_{h_1}^{h_2} \frac{A_{wp}}{C_0 A_0 \sqrt{2gh}} dh \tag{9-32}$$

For a definite time interval,

$$t_2 - t_1 = \frac{2A_{wp}}{C_0 A_0 \sqrt{2g}} \left(\sqrt{h_1} - \sqrt{h_2} \right) \tag{9-33}$$

Therefore, the total time, T, for an original difference in water level, H, for a rectangular tank is given by

$$T = \frac{2A_{wp}\sqrt{H}}{C_0 A_0 \sqrt{2g}} \tag{9-34}$$

On large-scale, expensive projects where pumping and flooding times are critical, model tests may be carried out to determine actual coefficients and system hydraulic characteristics. Tank levels may be monitored remotely by use of liquid level gauges, usually pneumatic manometer or electrical-resistance-type devices. Where it is not critical to know the precise level of interior water at all times, sounding holes or wells may be installed for periodic checking. Flood valves may be compressed-air, hydraulic, or manually operated valves. All systems should have a manual override capability in order to close valves in an emergency. The use of rising stem valve shafts allows operators to see at a glance and from a distance if valves are in an open or closed position. Flood inlets should be fitted with screens to keep out trash and debris.

Tank Ventilation

All interior spaces must be adequately ventilated to remove noxious vapors and reduce condensation. Natural ventilation relying upon air density, temperature differences, and wind usually is adequate for most uninhabited spaces. Ventilator openings are sized according to the tank volume and the number of air exchanges per hour required, using semiempirical relations. In some cases, the vent pipes may be used as safety valves to limit the depth of submergence of the structure. When the bottom of the vent pipe is effectively closed off by the rising interior water level, a volume of air equal to the length of pipe below the deck times the tank plan area is trapped and compressed by the differential head pressure of water inside and out. The final depth of the air cushion inside can be calculated for the desired minimum freeboard by using the ideal gas laws, assuming an isothermal process, which requires that the product of the pressure times the volume remains constant, or $P_1 V_1 = P_2 V_2$.

Tank Access

Tank access for inspection and maintenance usually is provided through watertight gasketed manholes with welded-steel vertical ladders. For very deep tanks, intermediate landings may be required. Manholes should be at least 18 to 21 in. in diameter. All sealed tanks and enclosed spaces must be tested for harmful vapors and adequate oxygen content before personnel entry because the corrosion process can consume all available oxygen, so manhole covers should have appropriate safety markings.

9.6 Transfer Bridges and Gangways

Access to shore for vehicles and equipment most commonly is provided by single-span articulated bridges or ramps of similar construction to highway bridges. Floating link-spans, as described in Section 9.1, provide a transitional transfer bridge arrangement at many Ro/Ro facilities. If the floating structure is far from shore, a series of bridge spans supported on intermediate pontoons forming a floating bridge, such as that shown in Fig. 9-2, may be used. Other forms of floating-pier access include sliding wedge structures, such as those used along steep riverbanks with large water-level variations and vertical lift-type bridges. Smaller, lighter spans limited to pedestrian access usually are called *gangways*. This section reviews some basic requirements of typical single-span access bridges and gangways. The reader is referred to Tsinker (1986) for further description of floating bridges and alternative access arrangements.

Transfer Bridges

Design of access-bridge structures, generally referred to as *transfer bridges*, follows the same principles as basic highway-bridge design, where the governing loads include the same vehicles and equipment in traveling condition as used in the pier-deck design. Other important design considerations include lightweight, free-draining decks with good traction; simplicity and ease of maintenance; and special attention to end connections and bearings. For long spans where the bridge dead load reactions are high, a separate pontoon may be used for the sole purpose of supporting the bridge so as not to affect the trim and capacity of the main pier. Alternatively, fixed adjusting towers with wire rope reeving and counterweights or other lifting mechanisms may be used, typically for direct transfer to a vessel, as shown in Fig. 9-15. The transfer bridge needs to be counterweighted to keep bridge reactions to the minimum required for positive grounding on the vessel or floating-pier deck. Fixed adjusting towers further allow the bridge to be lifted clear of the

Fig. 9-15. Transfer bridge with fixed adjusting tower for offloading of construction equipment and supplies from barges. Note wire rope reeving and counterweight system for operating bridge

floating structure when not in use, which may be beneficial for pontoon maintenance, under severe weather conditions, or for security reasons.

Access bridges may have hinge pin connections at both shore and pier ends, or at either end with the other end having a sliding bearing. Generally, a short transition plate or apron plate is required at both ends to span the gap created as the bridge pivots. Hinged bearings should be slotted to allow for some lateral motion, even though the bridge may be stayed to fixed points with relatively taut cables. Slotted, energy-absorbing, sliding bearings may be used for bridges subjected to longitudinal movements caused by vessel impact. At exposed locations where the floating structure is subject to notable heaving and pitching motions, the vertical separation force at the pontoon end must be calculated based upon the dynamic characteristics of the bridge, pontoon, and counterweight system if applicable. If it is found that the pontoon could accelerate downward faster than the bridge could follow because of its large angular inertia, a vertical restraint must be provided or system characteristics must be changed. Design guidance for movable and lift-type bridges and their mechanical equipment can be found in AASHTO (2007).

General design criteria for Ro/Ro ship-to-shore connection of interest in access-bridge design can be found in BSI (2007), ISO (1983), and ICHCA (1978). Fig. 9-16 illustrates some general geometric design criteria. Minimum roadway widths of 5 m (16.4 ft) and 9 m (29.5 ft) are recommended for one- and two-lane traffic, respectively (ISO 1983). Minimum vertical clearance under worst normal tide conditions should be 7 m (23 ft). The maximum bridge slope should not be steeper than 1:10 (5.7°) up or down with the pontoon at its maximum normal freeboard at the highest normal water level (e.g., mean high water or mean higher high water) or with the pontoon at its minimum normal draft at the lowest normal water level [e.g., mean low water (MLW) or mean lower low water (MLLW)]. The change in gradient between deck surfaces should be limited to allow small-wheeled cargo-handling equipment to cross the transition point without difficulty. In general, this limit requires a limiting angle of approximately 6.5°. Further recommendations can be found in BSI (2007) and ISO (1983).

Gangways

Gangways, sometimes referred to as brows, especially when used for direct ship-to-shore personnel access, may be constructed of timber, steel, or aluminum alloy. Marine-grade aluminum alloys are by far the most common construction materials, and prefabricated aluminum truss-type gangways are readily available in lengths up to 60 ft or more. Beam-type construction is usually suitable up to approximately 25-ft lengths. Walkway widths are usually from 2 to 6 ft, although for general purposes a minimum width of 3 1/2 ft is recommended. Maximum gangway slopes of 1:3 with the floating structure at its normal (MLW or MLLW) position and 1:2.5 at extreme low water usually are recommended in small-craft harbors (see general references for Section 9.8). Gangways typically are fitted with handrails a minimum of 3.5 ft high

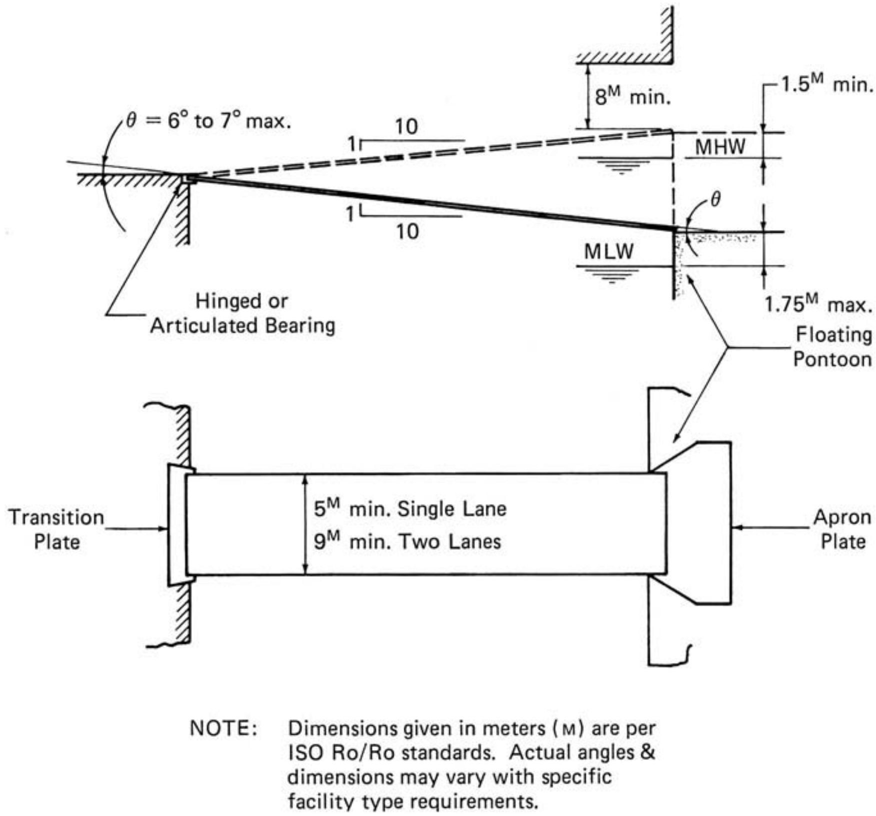


Fig. 9-16. Pontoon access-bridge design criteria

designed for a 50-lb/ft lateral force applied along the top rail. Shore-end connections should allow some freedom of movement in both directions. Bottom ends usually are fitted with wheels or rollers that ride on the pontoon deck. Gangways wider than 6 ft should be designed for a uniform load capacity of 100 lb/ft^2, and those less than 6 ft wide, for a minimum of 50 lb/ft^2 live load. NAVFAC (DOD 2005a) requires that all ship boarding brows, 3-ft minimum to 5-ft brows as required, be designed for a uniform live load of 75 lb/ft^2 and should be tested for a 150-lb/ft^2 live load. In addition, the brow is to be designed for an impact load, with one end being dropped 5 ft onto an unyielding surface. The maximum vertical deflection under drop of the brow is not to exceed 1/240 of the span. Gangway surfaces should be skid-resistant and preferably fitted with transverse cleats at 1-ft centers, approximately 1 in. wide by 1/4 to 1/2 in. high. For additional discussion of gangway design, see Section 7.6. The Americans with Disabilities Act (ADA), introduced in Section 7.6, provides ADA accessibility guidelines (ADAAGs) for recreational facilities (ADA 2002), which may have a profound effect on the design of access structures. Full compliance may be difficult to achieve, especially at certain vehicle ferry terminals and at locations with large tide ranges where the usual tide range may be accommodated but not the extremes. Discussion of these guideline regulations is beyond the scope of this text; however, the designer of subject facilities should keep informed on this subject and check on the most current editions of the regulations. See Appendix 3 for website addresses.

9.7 Mooring-System Design

A mooring system generally consists of the connection at the floating structure, a tethering cable, a strut or linking mechanism, the anchoring device or structure, and the soil mass that holds it. Moorings can be broadly categorized as fixed—whereby the structure is secured directly to a fixed pier, dolphin, or cantilever pile—or free, with the structure tethered via cables or chains to anchors in the bottom soil. The design of fixed structures was covered in Chapter 7, and some typical means of connection to fixed structures are illustrated in Fig. 9-17.

Fixed mooring connections must allow free vertical movement of the floating structure while keeping it securely in position. The connection also must permit some degree of list and trim of the structure. The simplest type of connection is the hoop-and-guide pile arrangement common in marinas. Spud and gripper arrangements commonly are used to moor floating dry docks and pontoon structures, and the gripper connection usually is articulated to allow for trimming and listing of the dock. Floating dry docks alternatively may be moored by a combination of spud guides and shear spuds, providing only lateral and longitudinal restraint, respectively.

Free moorings may be further classified as free-swinging, where a vessel is moored at a single point, usually to a buoy that supports the mooring chain or chains, or spread-moored, where a vessel or floating structure is secured from movement in both principal directions. Fig. 9-18 illustrates some free mooring arrangements. Almost all harbors have anchorage areas where vessels can anchor awaiting their turn at berth. At some locations, substantial single-buoy moorings are provided for this purpose. The use of SPMs at offshore deep-draft tanker terminals was introduced in Section 9.1. Conventional tankers or barges may be moored offshore or "in-stream" in rivers in a relatively fixed position to a series of mooring buoys, and their cargoes may be pumped or lightered ashore. This is one of the simplest forms of marine terminal. Floating piers, which invariably are shore-connected, can be primarily spread-moored but often have cables, chains, or struts connecting them to anchors alongshore at their shoreward end. Mooring chains usually are cross-connected, leading to opposite sides of the pontoon in order to allow deeper draft alongside. Tsinker (1986) and Tsinker and Vernigora (1980) provide further discussion of anchor cable arrangements. The general design of moorings for floating structures is also further addressed in BSI (1989).

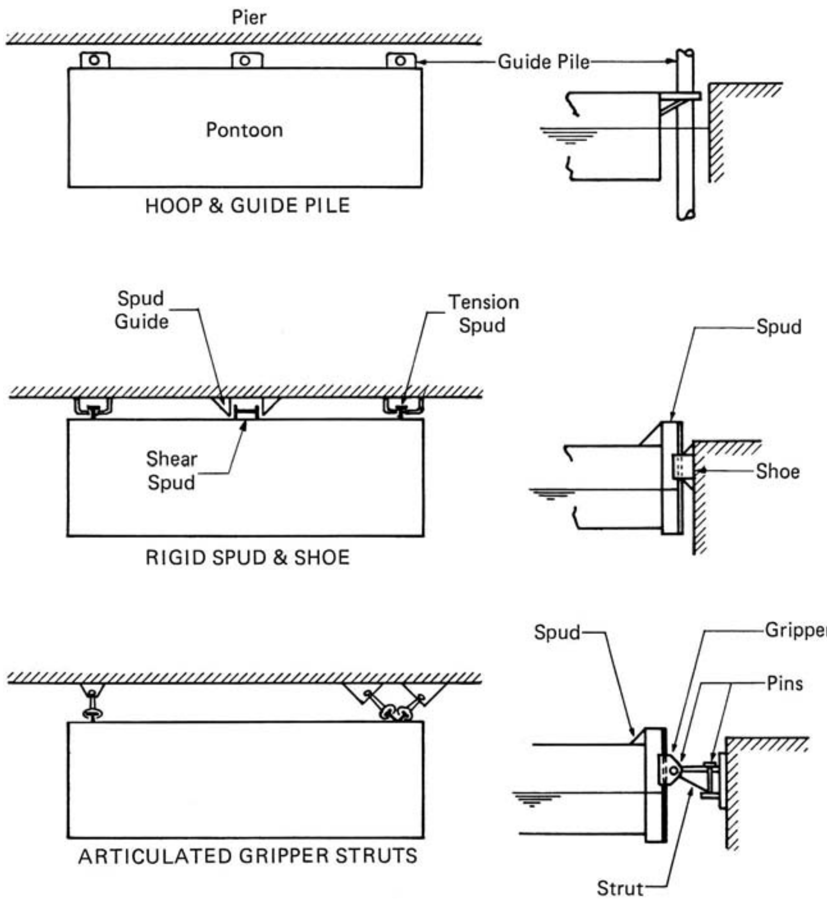


Fig. 9-17. Schematic of mooring connections to fixed structures

The remainder of this section is devoted to some basic principles common to the design of most free mooring systems. Particular attention is given to catenary chain mooring, with which all harbor engineers should be familiar. Wire rope and fiber lines also may be used as components of floating-structure moorings. Their general properties are covered in Section 6.2.

The Catenary Equations

A suspended chain or cable assumes a characteristic curve shape known as a catenary under its own weight. The catenary action of a mooring chain is beneficial in reducing the moored object's motions and in acting as a shock absorber in reducing peak mooring loads. Because a chain or cable only can act in tension, it produces a horizontal force, equal at any point along its length, that is a function of the chain's unit weight and its extension length. Fig. 9-19 illustrates basic catenary geometry,

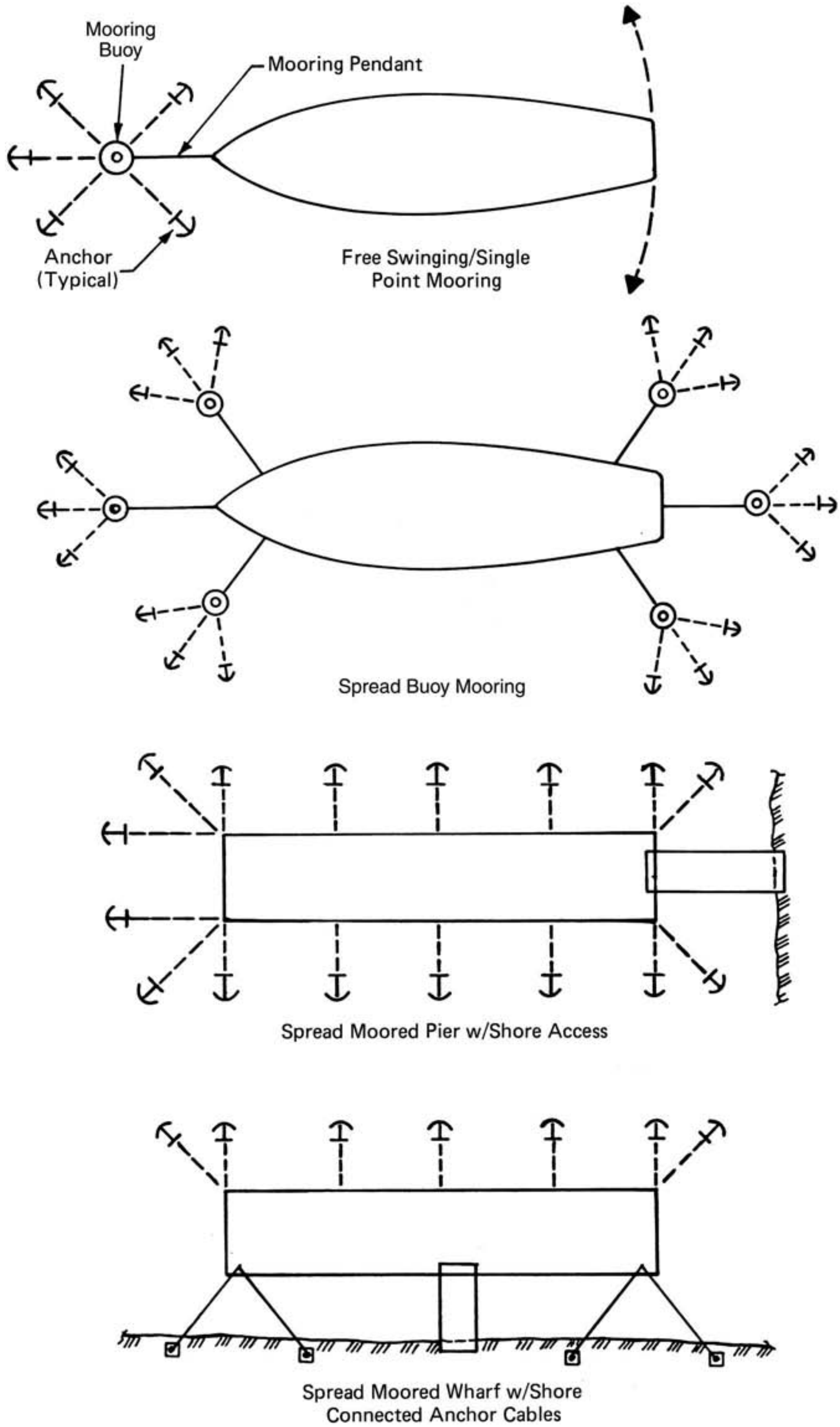


Fig. 9-18. Typical free-swinging and spread-mooring arrangements

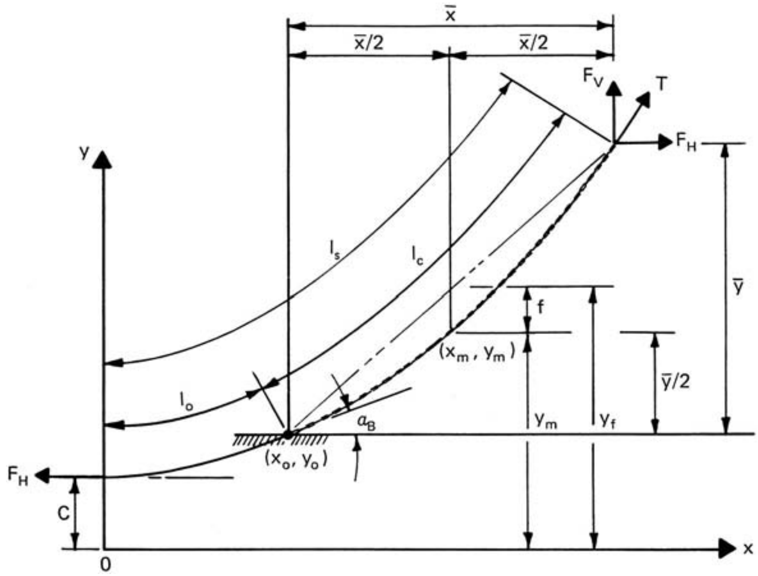


Fig. 9-19. Catenary geometry definition sketch

where the following equations apply:

$$F_v = w_s l_s \tag{9-35}$$

$$F_H = w_s c \tag{9-36}$$

$$T = w_s y \tag{9-37}$$

where

F_V, F_H, and T = vertical and horizontal force components and tension, respectively;

w_s = submerged unit weight of the cable; and

l_s = total curve length from the mooring connection to its hypothetical point of horizontal tangency and intersection with the y-axis.

Note that the curve shown actually is only one-half of a true catenary, which would continue symmetrically to a suspension point at $(-x,y)$. The distances c and y are constants of the curve. The constant c is the distance to the y-intercept and is equal to the horizontal force divided by unit weight of the chain. The larger the chain tension, the larger the value of c. Note that if x were reduced such that T became tangent at y, then c would equal 0, and $F_H = T$ would be tangent at the origin. We are usually interested, however, in only a portion of the catenary acted upon by a horizontal force (F_H), where it assumes some angle (α) with the bottom at the anchor point. For the case of zero slope at the anchor, the cable length, l_c, equals l_s. In general, it is desirable to keep α minimal; in particular, when drag-type anchors are used, α should be 3° or less under design load conditions.

The following equations are used in catenary computations:

$$y^2 = l_s^2 + c^2 \tag{9-38}$$

$$y/c = \cosh\left(\frac{x}{c}\right) \tag{9-39}$$

$$l_s/c = \sinh\left(\frac{x}{c}\right) \tag{9-40}$$

$$\sqrt{l_c^2 - y^2} = 2c\sinh\left(\frac{\bar{x}}{2c}\right) \tag{9-41}$$

$$\frac{y}{l_c} = \tanh\left(\frac{x_m}{c}\right) \tag{9-42}$$

$$x = x_m + \frac{\bar{x}}{2} \tag{9-43}$$

Further instruction in the use of the catenary equations is provided by NFESC (DOD 1999).

It often is desirable to know the cable sag, as for checking vessel keel clearance. The sag (f) from a taut line measured vertically at $\bar{x}/2$ is given by

$$f = \frac{w_s(\bar{x})^2}{8F_H} \tag{9-44}$$

Weights called sinkers often are attached to mooring chains near their midlengths in order to reduce the angle at the anchor and sometimes also to increase under-keel clearances for berthed vessels. In this instance, the final chain geometry is calculated by assuming the intersection of two catenaries at the sinker location where the submerged weight of the sinker is taken as an equivalent length of chain. A series of clump weights may be used in lieu of a single larger sinker. Sometimes wire rope and/or fiber lines may be used in combination with chain. Such systems are referred to as multicomponent mooring systems. A direct solution of the behavior of such systems is not possible; iterative techniques with assumed equilibrium positions must be followed. Ansari (1979) gives an overview of multicomponent mooring systems design. In deep water and where strong currents exist, the hydrodynamic resistance of the mooring cable becomes important. The book by Berteaux (1976) and tables by Pode (1951) are useful in such applications.

General Design Criteria

Mooring-system design criteria and loads on floating port structures are determined in a manner similar to that described for berthing and mooring loads in Chapters 5 and 6. Free moored structures, however, have greater freedom of motion than rigidly moored structures, so a wider range of dynamic response to environmental forces is possible with free moorings. Headland et al. (1989) performed a dynamic analysis of a spread-moored floating dry dock under various combinations of steady and time-varying wind and regular and irregular waves and concluded that the presence of an unsteady wind field gives considerably greater peak loads than those given by a static wind analysis (see also Sections 6.5 and 6.9). Also, the relatively low natural frequencies of spread-moored systems make them prone to high loading under low-frequency excitation. Wind spectra (see Section 6.5) have been developed for offshore structure design applications (Ochi and Shin 1988), and means of calculating the effects of low-frequency vessel motions on offshore drilling and semisubmersible units have been given by the American Petroleum Institute (API 2005). Kwan (1991) presents an overview of design practice for floating production systems (FPSOs).

In general, for chain moorings, the load in the mostly highly loaded chain should not exceed 33% of the minimum breaking load (MBL) of the chain under maximum design operating conditions or up to 50% to 60% of MBL under extreme environmental conditions; see DOD (2005b) for specific U.S. Navy design requirements. This load is exclusive of any corrosion allowance, and further reduction should be taken in cases where the chain is subject to sharp changes in directions, such as at fairleads and hawsepipe connections. In many instances, however, the chain may be sized for its weight to help limit excursions under load with tidal variations, and much greater factors of safety are realized. Anchor efficiencies drop off greatly with increased chain angle, and limiting vertical angles between the chain and seabed usually are within the range of 3° to 6° above the horizontal. Floating-pier movements are a function of water depth, tide range, and mooring-system stiffness, and allowable surge/sway movements usually are determined by access-bridge requirements. The mooring chain scope also affects relative movements with tide changes and bottom chain angles. Although a greater scope is favorable, it must be traded off against cost and spatial restraints. Minimum scopes are usually on the order of 3:1 to 5:1, unless substantial uplift resistance can be provided at the anchor.

Although some floating piers may be considered to have shorter design lives than the nominal 25 years often assumed for the design of fixed structures, it is still recommended that extreme conditions associated with the 100-year event or a 0.999 probability level be designed for, with the exception of temporary or expendable structures intended to be removed from the site in less than 1 to 2 years, for which a 25- to 50-year recurrence interval or less may be appropriate.

Mooring-System Components

Mooring-system components include anchors, chains and/or cables, connecting hardware, sinkers, and structure end connections. The structure end connections should allow a convenient means of adjusting and resecuring the mooring chains, but at some installations where anchors can be accurately located, chains may be connected directly to fixed mooring eyes on the pontoon hull or deck. Chains may be led around submerged fairleads at the bottom of the pontoon and up to the pontoon deck, or directly to the deck over a fairlead and chafing plate, and may be connected to pad eyes directly or to a chain "wildcat," windlass, or tackle arrangement for ready adjustment. Alternatively, chains may be led through the pontoon hull and secured with chain claws that straddle the chain link and hawsepipe opening. Mooring hardware includes shackles, connecting links, equalizer plates, end connectors, and so on, whose description is beyond the scope of this text. The reader should consult DOD (2005b) and Vervloesem (2009) for a detailed description of mooring chain and wire rope hardware and its application. API (2014) provides guidance for the use of synthetic fiber lines in offshore mooring applications and compares their properties with wire rope and chain mooring systems.

Chain is either of open link or stud link construction, of mild carbon or high-alloy steel, depending upon strength requirements. Open link chain normally is made by automatic electric welding processes, whereas stud link chain may be made by welding, forging, or casting. Open link chain is classified typically as proof coil, buoy, BBB, conveyor chain, and so forth, according to its link proportions. Open link chain commonly is used in smaller sizes, below 1 in., but stud link chain is preferred for permanent moorings and is almost universally used in sizes of 1 in. and greater. Stud link chain does not twist and kink like open link chain and is preferred in applications where the chain may go slack or pile up on the bottom at some point in its load cycle. Chain strength is based upon actual break tests of a sample, and individual lengths (usually 90-ft shots) are certified by proof tests of around two-thirds of breaking strength. Working load limits may be applied to chains and are usually on the order of one-half of proof load. Complete tables of chain strengths and weights are published by the various chain manufacturers.

The ultimate strength of a chain link is approximately 1.55 times the theoretical tensile strength of one of its side wires, and the elastic stretch of a ferrous stud-link chain is approximately 2.65 times the elastic stretch of one of its side wires, resulting in an elongation of about 1% at proof load. According to API (2005), the elasticity (T/δ_c), where T is the chain tension in pounds and δ_c is the stretch in feet, is given by

$$T/\delta_c = 1.2 \times 10^7 D_c^2 / l_c \tag{9-45}$$

where

D_c = nominal diameter (wire size) in inches, and

l_c = chain length in feet.

The submerged weight of chain is equal to 87% of its weight in air. The general engineering properties of wire rope and fiber lines were introduced in Section 6.3.

Anchors

Anchors can be loosely classified according to their primary mode of developing lateral resistance within the bottom soil. Fig. 9-20 illustrates representative anchor types.

Gravity-type anchors, such as concrete blocks, depend on their submerged weight and the effective coefficient of friction with the bottom soil, which for a concrete

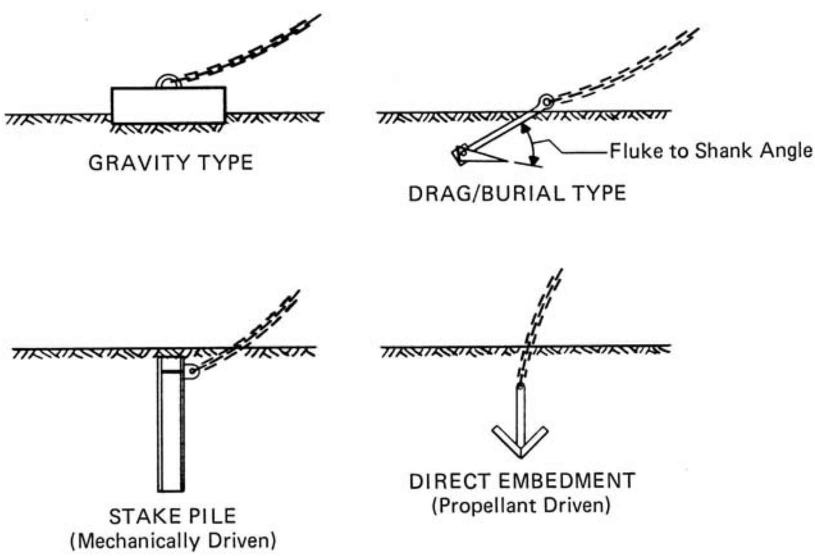


Fig. 9-20. Classification of basic anchor types

block usually is assumed to be between 0.5 and 1.0. Thus, its efficiency or ratio of horizontal holding power to its dry weight in air is on the order of 0.25 to 0.5. Such massive objects may gain some added resistance caused by bottom suction in cohesive soils and passive soil resistance if well embedded, as by jetting into the bottom. Pekin et al. (1987) conducted experiments on gravity anchors in sand for large-scale applications. Anchor chains themselves contribute to the holding power of a mooring; according to API (1996), the holding power of a chain or wire rope resting on the bottom is equal to the length of cable on the bottom times its submerged weight per unit length, times a friction factor of 1.0 for chain and 0.6 for wire rope. Gravity anchors also may be pinned to the bottom with short needle piles driven through the anchor block or via projections cast into their bottoms. Gravity anchors may be cast in shapes that help them to penetrate the bottom and gain additional passive resistance within the soil, such as the self-burying type described by Tsinker and Vernigora (1980).

Drag-type anchors must be dragged into position to set their flukes and gain holding power by means of activating shear stresses within the bottom soil. Drag-type anchors can also be divided into traditional *ship-type anchors*—such as the mushroom, kedge, and stockless, which hook into the bottom rather than penetrate it and thus still depend upon their weight for holding power—and contemporary *burial-type drag anchors*. Holding-power ratios range from around 2.0 to 3.0 for mushroom types up to 5.0 to 7.0 for stockless types under the most favorable soil conditions. Contemporary ship anchors are mostly of the burial type, designed to penetrate the bottom and engage as large a portion of a soil mass as possible.

The capacity of burial-type anchors depends upon the soil type, depth of penetration, fluke area, fluke-to-shank angle, and the anchor's overall shape and configuration as they affect its stability and ability to penetrate the soil. Holding power ratios of 15 to 60 or more times the anchor weight may be attained. DOD (1999) contains much physical and descriptive data, and NCEL (1985) contains comparative data on anchor holding power and means for calculating soil resistance for contemporary burial-type anchors. Carchedi et al. (1984) carried out experiments on a generic burial-type drag anchor in sand with variable fluke angles, shank lengths and shapes, and surface roughness for different soil densities and bottom slopes. They found anchor holding power to be directly dependent upon the depth of burial, which is in turn dependent upon soil properties, bottom slope, and anchor configuration. Sandy soils generally are considered to be good holding ground because of their high friction and easy penetration, but anchors must be stabilized against rolling over as they approach their tripping moment, where they rise on the tip of the flukes during burial. The optimum fluke-to-shank angle in sand is considered to be 32° and for silts and mud, 50°. Mud also may provide a good holding bottom for anchors with large fluke areas. Stiff-penetrable clays provide good holding, whereas softer clays may be disturbed during penetration.

Embedment anchors are forcibly driven into the bottom, either by underwater pile hammers or surface equipment with followers or by propellants, such as explosives.

Stake piles usually are driven with chains attached and provide excellent holding power where site conditions permit. In addition, they can be accurately located, unlike drag anchors, which must be dragged an uncertain distance before they "fetch up." Reese (1973) presents a design methodology for anchor piles. Other embedment types include umbrella piles, which open their flukes upon reaching the required penetration depth, and the vibratory type (Shaw 1972), which vibrates itself into the bottom like a vibratory pile hammer. Helix-type anchors, which have helical fins around a steel shaft, can develop high pullout resistance. This type of anchor is commercially available and has become increasingly popular for small-craft moorings. The suction-pile-type anchor is driven into the bottom by pumping water from within its cylindrical body, which is open at the bottom and sealed at the top.

The final choice of the best anchoring scheme for a given project depends upon the nature and depth of the bottom material, holding power required and scope as it affects chain slope, need to accurately locate anchors or limitations on dragging anchors into position, need to recover anchors, and first cost and long-term maintenance. General discussion of anchors and anchoring techniques is provided by Hinz (1986), oriented toward small craft, and Puech (1984) for offshore operations.

Free-Swinging Moorings

Free-swinging or single-point moorings allow a vessel to "weather vane," bringing itself into alignment with the wind and/or current, thus generally minimizing its total mooring load. Preliminary designs of free-swinging moorings traditionally have been prepared by using static analysis methods, with final designs based on physical model tests. Recent progress in numerical analysis of single-point moorings, however, has led to an increase in the use of such models for both preliminary and final designs. A brief description of static and dynamic analysis methods is provided in the following paragraphs.

Static-analysis methods presume that a vessel secured by a free-swinging mooring assumes an equilibrium position under wind and current loading. Though wind and current can be coincident, the largest mooring loads generally occur under oblique wind and current attack. In calculating pendant loads, account must be taken of the increased projected area with yaw angle and of the horizontal and vertical components of the mooring pendant. When wind and current act together, the vessel lies at an odd equilibrium position that can be found only by trial-and-error calculations. It is noteworthy that the maximum mooring force for combined wind and current may not occur with the maximum wind or current speed.

Experience both in the field and in physical model tests has shown that vessels secured to single-point moorings often do not assume an equilibrium position. Instead, vessels tend to oscillate around the equilibrium position even under steady wind and current attack. This phenomenon, commonly called *fishtailing* or *kiting*, can result in mooring loads considerably higher than those computed by static-

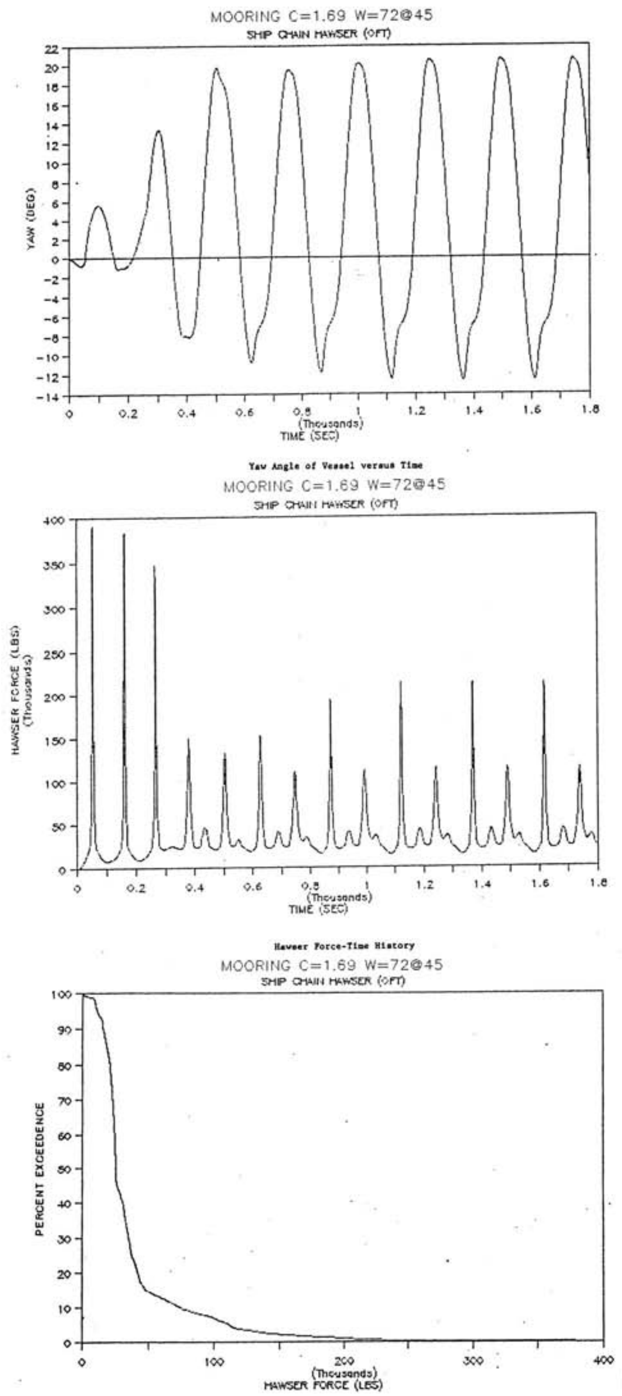


Fig. 9-21. Example computer output for 7,000 displacement tonnage (DT) vessel at single-point mooring subject to combined wind and current

Source: Courtesy of John R. Headland, P.E.

analysis methods. Numerical methods for the dynamic analysis of single-point moorings are based on the time-domain approach discussed in Chapter 6. Wichers (1988) presents a good summary of time-domain analysis for single-point moorings, classifies single-point mooring response as either dynamically unstable or dynamically stable, and presents a criterion for establishing stability conditions under wind and current attack.

Fig. 9-21 summarizes typical results from a computer simulation of a naval vessel secured to a single-point mooring under wind and current attack, presenting time histories of vessel yaw and mooring hawser loads as well as a plot of the probability of exceedance of hawser loads for the simulation. It should be noted that the yaw time history reflects a fishtailing period of roughly 4 1/4 minutes.

Particular caution must be exercised in relatively shallow water at sites exposed to ocean swell or long-period waves where surge motions may be significant. Note, too, that design mooring conditions vary with the intended use of the system, for example, occasional transient or harbor of refuge under storm conditions.

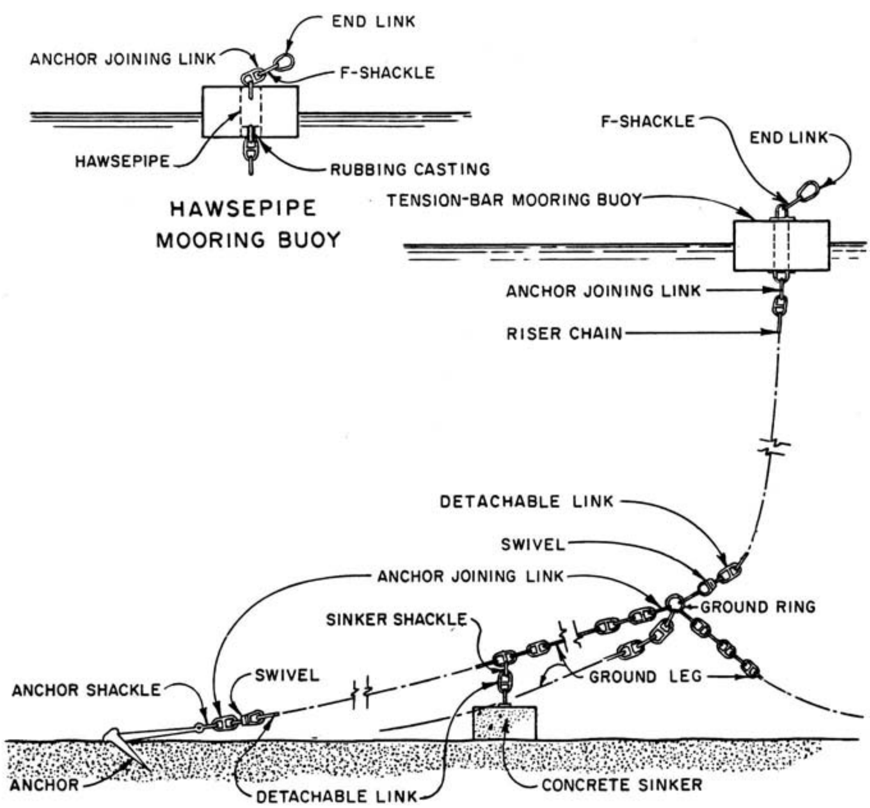


Fig. 9-22. Representative free-swinging, riser-type mooring

Source: Adapted from DOD (1999)

The U.S. Navy classifies standard moorings for given holding power and swing radius conditions (DOD 1999). The watch circle described by the vessel's swing radius takes up a great deal of space, as described by Gaythwaite (1989) and in the following discussion on moorings for yachts and small craft.

A typical mooring arrangement usually consists of a single stake pile or gravity anchor or an arrangement of three to six drag-type anchors in a radial pattern connected to a heavy bottom chain and thence to a riser chain via a ground ring or spider plates and a mooring buoy, which supports the weight of the chain and transmits the load to the vessel's mooring lines. A representative free-swinging, riser-type mooring is illustrated in Fig. 9-22.

Some buoys are fitted with their own, usually braided nylon, pendants supported by smaller pickup buoys to facilitate pickup by the vessel. Contemporary mooring buoys usually are constructed of a rigid, closed-cell foam core encapsulated in an elastomeric or fiber-reinforced shell with a central tension member or hawse tube arrangement for connecting the mooring chain. Commercially made mooring buoys are available with up to approximately 100 tons of pull-through load capacity, with net buoyancies of 25 tons. Mooring buoys alternatively may be constructed of steel plate with internal watertight compartments for damage control. The minimum freeboard under load should be on the order of 18 in. Other buoy applications include marker buoys (usually spherical), utility floats, chain or pendant buoys, and navigation buoys.

9.8 Marinas and Small-Craft Facilities

Floating-Dock Systems

Floating docks are a prominent feature of most marinas with tide ranges or seasonal water level changes of 3 to 4 ft or more and are almost universally used where the tide range exceeds 5 to 6 ft. Floating docks also are commonly used as landing floats and increasingly as floating breakwaters. Floating-dock systems may be moored by cantilever piles, piers, or pile groups, struts to fixed points along the shore, or cables or chains to underwater anchors. A common design requirement for marina floats is that they maintain some minimum freeboard or change in draft under a uniformly distributed live load, often given in the range of 20 to 30 lb/ft^2. At rough-water locations, this requirement should be given special consideration to ensure that adequate reserve buoyancy is obtained.

Design live loads in the range of 25 to 50 lb/ft^2 should be considered at exposed sites, and live loads of 50 to as much as 100 lb/ft^2 for large public landing floats, commuter boat service, and commercial small-craft facilities. Corresponding minimum freeboards under full live loadings are usually within the range of 8 to 16 in., and greater for live loads exceeding around 40 lb/ft^2. Concentrated live loads on the order of 500 lb acting anywhere also must be accommodated by the float's structure. For solid prismatic-type floats, the wave bending moment for design waves of 1 ft or

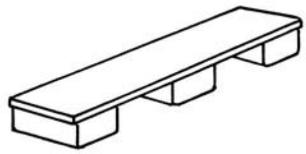
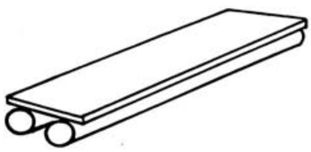
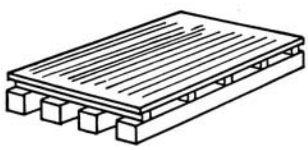
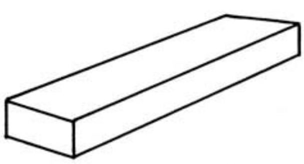
FLOAT TYPE	TYPICAL CONSTRUCTION	BEHAVIOR IN WAVES
UNITIZED FLOATATION 	Metal or wood deck frame w/ wood decking supported on individual buoyancy units, usually of closed cell foam encapsulated in rigid plastic shell.	Often springy underfoot and lively in waves. Relatively low mooring and conn. loads. Good survivability in waves if soundly built. Low reserve buoyancy.
CATAMARAN 	Metal deck frame w/ metal or wood decking supported on cont. buoyancy tanks often of metal pipe sections w/ floatation fill.	Rel. solid underfoot. Mod. to low motion resp. in waves. Moderate mooring & conn. loads. Cross structure req's. careful design. Mod. to low reserve buoyancy, may lose righting moment rapidly w/ heel angle.
RAFT 	Wood deck over bolted wood frame supported on heavy timbers laid parallel & w/ additional buoyancy provided by rigid foam billets battened between timbers.	Rel. massive, solid underfoot. Timber flexes allowing good survivability in waves if well constructed. Billets may wash loose under wave action. Moderate mooring and conn. loads and reserve buoyancy.
SOLID 	Monolithic, box hull of steel or reinforced conc. construction w/ foam fill.	Solid underfoot, slow motion response, massive w/ rel. large mooring & conn. loads which may become problem under sustained wave action. Monolithic hull is inherently strong. Excellent reserve buoyancy.

Fig. 9-23. Comparison of generic floating-dock types

Source: Adapted from Gaythwaite (1989)

greater height usually is much larger than the still water bending moments associated with concentrated live-load requirements. Torsional stresses in waves also may be critical, especially for landing floats with large beam-to-length ratios (B/L_s). Equivalent lateral loads for design caused principally by wind on berthed vessels and a contingency for normal berthing and mooring loads usually range from 60 to 120 lb/ft for most marina applications. At exposed locations and for commercial small craft, these loads may be much higher. Gaythwaite (1989) reviews general design criteria for small-craft facilities at exposed locations.

There is a large variety of commercially available floating docks on the market. Fig. 9-23 illustrates some typical generic types and their usual types of construction and behavior under wave action. PIANC (1997) provides a detailed review of floating-dock design standards. The layout and design of marinas and small-craft facilities in general is covered in ASCE (2012), DOD (2009), and Tobiasson and Kollemeyer (2000).

Small-Craft Moorings

Free-swinging moorings are a common and expedient means of mooring yachts, especially at exposed locations and sites where the construction of marinas is not feasible. The general geometry of a typical yacht mooring is illustrated in Fig. 9-24. Moorings usurp a relatively large water area, so a central focus of mooring design is to minimize the swinging radius or watch circle at low water while providing adequate scope (S_c), the ratio of chain length to water depth, under high water conditions. The minimum value of S_c should be based upon an appropriate design water level, considering maximum seasonal water levels or storm conditions. It is

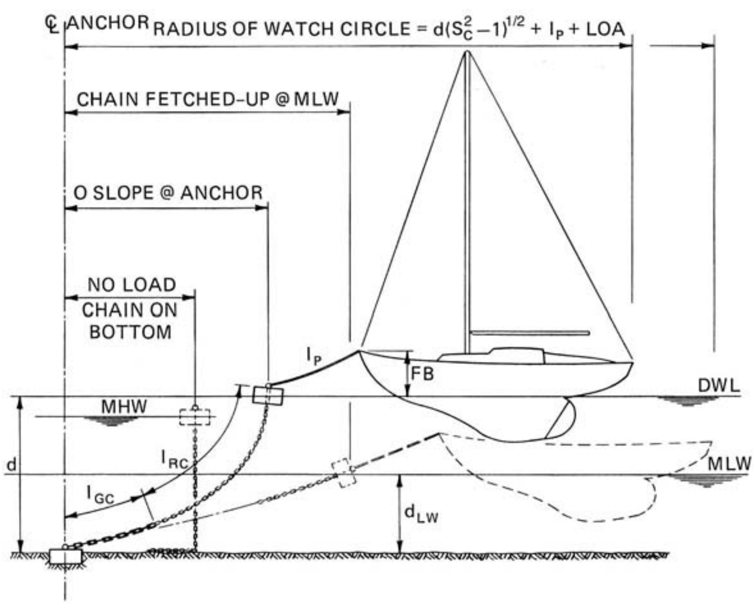


Fig. 9-24. Free-swinging yacht mooring geometry

Source: Adapted from Gaythwaite (1989)

desirable to maintain a condition of zero slope of the mooring chain at the anchor under such conditions. The watch circle radius is equal to the sum of the water depth times $\sqrt{S_c^2 - 1}$ for the chain in the fetched-up condition, plus the pendant length, plus the vessel length, plus any desired safety margin or contingency. The maximum value of S_c should be taken at MLW or MLLW. At many locations, where the water area is at a premium, less than the full swinging clearance may be provided. In such cases, it is essential to place vessels with similar hull shape characteristics together and to isolate dissimilar types (e.g., deep-draft sailboats and high-superstructure power boats respond differently to variable wind and current combinations). Where this separation is not possible, fore and aft moorings or other restrictive mooring arrangements may be used. PIANC (2002) provides a review of various alternative mooring systems for small craft.

In laying out new mooring areas, the engineer can design for classes of moorings with a given holding power and watch circle radius based upon site-specific design wind speeds, water depths, and so on. Vessels then can be classed according to their holding power requirements and swinging characteristics, in terms of their displacement or other characteristics that correlate with wave surge loads and wind area as well as the length overall (LOA). In areas affected by currents, an equilibrium position must be calculated by trial and error, based on relative current and wind directions. It is likely to be found that vessels lie at odd angles and that the resultant mooring force is not a simple addition of wind and current vectors.

Wind loads are calculated by using the drag-force equation on the forward-projected area yawed to 30° to get the longitudinal component of wind force, and then by applying a surge factor (C_{SF}) to account for wind gusts and swinging and for vessel surge, pitch, and heave motions in waves. The surge factor is normally within the range of 1.25 to 2.0 (Hinz 1986), depending upon the degree of exposure. Drag-force coefficients typically range from 0.6 to 1.2. Caution should be exercised at certain shallow water locations where long-period waves may be present because the wave particle orbits become more elliptical and may result in large surge, fore, and aft motions. Van Dorn (1993) provides useful information for calculating wind loads on sailing yachts and also an excellent discussion of the dynamics of anchored yachts. The American Boat and Yacht Council (ABYC 1978) gives recommended permanent mooring capacities for pleasure boats ranging from approximately 1,500 lb for a boat of 25-ft LOA to 6,000 lb for a vessel of 60-ft LOA.

The use of chains is preferred for permanent moorings because of their weight and catenary action, strength, and abrasion resistance. In areas of large tide range, it is recommended that a short length of heavier ground chain be used, permanently secured to the anchor block and of sufficient length to be brought to the surface at low tide for periodic replacement of the smaller riser chain, which terminates at the mooring buoy. Mooring buoys can be designed to support the full weight of chain at high water, or at relatively shallow water sites, the pendant simply can be secured to a pickup buoy, which need only support the submerged weight of the pendant. The pendant should be made of nylon line for its shock-absorbing characteristics and should preferably be of a length two to three times the height of the vessel's bow chocks above water. The pendant should be wrapped with chafe protection where it passes through the vessel's chocks. Twisted nylon generally possesses better chafe resistance than the braided types.

Galvanizing a chain, which typically reduces its strength about 10%, may add from 6 months to perhaps 1 year or more to its life, after which time the zinc coating goes into solution in seawater. The relative cost of galvanized chain should be compared with the cost of an extra corrosion allowance on ungalvanized chain.

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CHAPTER 10

Introduction to Dry Docks

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Dry docks are structures that allow complete dry access to the underwater portions of a vessel's hull for maintenance, overhaul, and repairs. Dry docks can also be used for new construction and launching. Dry docks can also provide a means of transferring vessels to or from dry land for temporary storage or lengthy overhauls, or they may be used for the launching of newly constructed vessels. They are the workhorses of ship repair facilities and may be used in lieu of traditional building ways at shipyards devoted to new construction.

There are various types of dry docks, including those that physically lift the ship from the water, such as floating dry docks, marine railways, and vertical-lift systems, and traditional basin dry docks that dewater an enclosed space around the vessel. This chapter is intended as an introduction to the various types of dry docks and their basic principles of design and operation. The first section presents a brief review of the capabilities and major features of each type of dry-docking facility and factors affecting the selection of the appropriate facility. Following sections are devoted to the basic design of the principal dry dock types. The final section deals with mobile straddle lifts and other miscellaneous means of dry-docking small craft.

10.1 General Characteristics, Features, and Factors Affecting Dry Dock Selection

There are four basic types of dry-docking facilities: graving or basin docks, marine railways, floating dry docks, and vertical lifts. Vertical lifts may be further divided into two groups: the ship lift and the mobile marine lift.

Basin dry docks are excavations with a gate at one end opening to the waterway. Marine railways consist of a cradle that travels on inclined tracks to launch and retrieve vessels. Floating dry docks are structures capable of sinking below the water's

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surface to receive a vessel and lift it out of the water. Vertical lifts are ship elevators designed to be lowered into the water to receive a ship and then lift it clear of the water.

Several characteristics and features are desirable for the efficient operation of a dry dock, regardless of the type of facility chosen (Crandall 1964, Cornick 1958):

1. Adequate space is necessary in and around the dry dock for ease of personnel and material movement with a vessel in the dock.
2. Fast and efficient access is needed to and from the dry dock and the vessel in the dry dock. Access for vehicular traffic is desirable. The introduction of traveling staging and mobile manlifts for use around the docked vessel has helped to make ship repair more efficient.
3. Adequate light and ventilation are necessary to ensure good working conditions.
4. Support facilities are required, such as electrical and mechanical services. Outlets for water, compressed air, steam, and welding and cutting gases often are located at service stations for ready access to the vessel. The service stations usually are placed along the wing and pontoon decks of floating docks, in recessed galleries or at the coping of basin docks, and in surrounding ground areas for marine railways and lifts. Electrical outlets are provided to supply power to the vessel in the dry dock and for repairing the vessel. Ship services and utilities are discussed in Section 7.8.
5. Material handling systems are needed for heavy items and equipment. Cranes are an essential dry dock feature, making ship repair and outfitting efficient. Cranes may operate on the wing walls of a floating dry dock or on the adjacent pier and/or land area to service all types of dry docks. More discussion of cranes may be found in Section 4.3 and Mazurkiewicz (1980).
6. An efficient method should be provided for moving a vessel in and out of dry dock. Docks often are equipped with tensioning winches, capstans, and other line handling hardware, which allow the dry dock crew to control the vessel as it enters the dock and position it correctly over the blocks.
7. Electric or electrohydraulic capstans are most often used to handle the lines. Some of the larger basin docks and floating docks use centering/haul-in trolleys that travel on monorails fitted along the inboard side of each sidewall. These trolleys use wire ropes pulled by winches that can center the vessel in the dock and haul it into its docking position.
8. A proper blocking system must be provided. Blocks are used to support the weight of the ship while positioning it at a convenient height to provide work access underneath and leave much of the bottom area free for cleaning, repair, and painting. They also provide stability to prevent the ship from tipping over because of high winds or earthquake forces. The blocking can be considered as a mattress that provides support yet yields elastically to account for irregularities in the fit of the ship.

The most common blocking scheme uses a row of fixed, prepositioned keel blocks with a row of stabilizing bilge blocks on each side. The bilge blocks may be prepositioned or brought into position after the ship's keel has touched the keel blocks. The latter case is preferred if the ship has considerable shape or where clearance for sonar domes or propellers is needed to permit the ship to enter the dock. Many systems have been developed for this purpose, including electrical, hydraulic, and compressed air systems, and chains pulled by winches. Because of its simplicity and durability in a marine environment, the chain/winch system is most often used.

Most large blocks are made of composite construction with concrete or steel bases and a timber or rubber cap piece to provide the necessary cushioning against the ship's hull. Timber may be added to the base of the block to provide cushioning against irregularities in the dock floor.

Removal of blocks to provide access to hull areas in need of repair is accomplished through the use of sand frames set on top of the blocks' concrete base, as shown in Fig. 10-1. A confined layer of sand in the frame supports the top pieces of wood. Provision is made to remove the sand by water jet, allowing the wood to drop so that the block can be removed. Other, more sophisticated systems are available for block removal, but this simple system is perhaps the most common and successful one. More information on blocks and blocking concepts can be found in Crandall (1987a, b).

Fig. 10-1. Block with sand frame for easy removal

Selection of the appropriate type of dry-docking facility is influenced by many factors, including the following:

1. The dimensions, weight characteristics, and general features of the vessels to be serviced by the dry dock.
2. Conditions at the site of the dry dock and the associated land facilities, including available land area, available area in the water, proximity to navigable channels or open water, tides, currents, topography, and soil conditions.
3. The purpose of the dry dock. Is it used solely as part of a new building program, for long-term vessel repairs, for short-term vessel repairs, or for some combination of all these purposes?
4. The near- and far-term goals of the shipyard and the potential future extension of the dry-docking facilities.
5. Financing. Investors may be more willing to provide working capital for a dry dock facility that can be more easily relocated (such as a floating dock) than another type.

These considerations are discussed as they relate to each type of facility in the sections that follow. Additional discussion of these factors can be found in Crandall (1964) and Salzer (1986a).

Factors that affect the operational aspects of dry-docking include characteristics of the ship, features of the dock, and, in some cases, the two together (Crandall 1987b).

A ship is necessarily strong to resist stresses encountered at sea. The stiffness that allows the vessel to resist excessive flexing in waves also results in a distribution of weight concentrations because of machinery, fuel, cargo, and so on, to fairly uniformly load the keel blocks along its length when it is dry docked. The ship-hull girder stiffness usually is sufficient to permit overhang of the ship beyond the block length both fore and aft. As a general rule, the safe overhang at either end (or both ends) for an unloaded vessel may be up to one and one-half times the molded depth of the ship (Crandall 1976). Fig. 10-2 shows a floating dry dock with a large vessel overhanging at one end.

Ships are designed to be stable, floating upright in the water for a variety of loading conditions. Although the vessel is usually in a stable condition when it arrives for dry-docking, its stability characteristics change (become less stable) as it sits down on the blocks. The vessel must be firmly cradled by the side blocks before it loses the stabilizing effect of the water. The point at which this happens can be calculated to ensure that the side blocks can be brought to bear against the hull before instability occurs. Once in dock, the only way to ensure that the vessel will be stable upon refloating is to keep careful track of all weight changes on the vessel and calculate its metacentric height (GM), a measure of stability, before undocking. Most accidents attributed to instability occur during the undocking of the vessel because the

Fig. 10-2. Floating dry dock with large vessel overhang

Source: Photo courtesy of Crandall Dry Dock Engineers

changes in weight have not been monitored and the new stability characteristics were not recalculated (Heger 2005).

The strength properties of the dry dock must be considered before dry-docking a vessel. The load distribution through the dock structure into the foundation or buoyant support must be considered. For a floating dry dock, stability of the dock itself must also be sufficient at each phase of lifting the vessel out of the water. A heavy load concentration, or a ship that is too heavy, can cause damage to a dry dock's structure and/or its foundation. A dock's safe lifting capacity should be assessed to determine the maximum total load and load per unit length that can safely be put on the dock. These limits should be certified by an engineer knowledgeable in the design of that type of dry dock and should never be exceeded (ASCE 2010).

The total weight of the ship and its distribution on the blocks must be known to ensure that the load ratings are not exceeded. A review of methods for calculating dry dock block reactions is presented by Taravella (2005), and Jiang et al. (1987) present a computer program for predicting dry dock block reactions. An investigation of keel block pressures when docking an aircraft carrier is presented by Palermo and Brock (1956), and Yeh and Ruby (1952) present an alternative method of calculating keel block loads. A simplified method for determination of the ship's approximate weight distribution and general discussion of keel block loads and resulting stresses in floating dry docks is presented in Section 10.4.

A major difference between a floating dry dock and other kinds of dry docks is that the forces between the blocking and the ship are a function of the floating dock's buoyancy as well as the ship's load. These forces can be somewhat controlled, however, by adjusting the buoyancy through variations in the dock's ballast-tank water levels. As with all docks, floating dry dock capacities are given in terms of total load and load per unit length. The stability of the ship–dock system is a critical factor and must be considered in the design and the operation of a floating dock. Vertical lifts have no inherent questions of stability, but adequate strength is critical because a failure can lead to a free-gravity fall of the ship. The ship's weight is distributed by the platform to the peripheral lifting means—wire ropes, chains, jacks, and so on. Depending on the ship's position on the platform and its weight distribution, some hoists carry more load than others. A lift's ability relates to the total capacity and the maximum load per unit length, which is a function of the number, size, and spacing of the lifting units. Ship lifts have been built as large as 28,000 tons lift capacity, and their capabilities have increased steadily over the years.

10.2 Basin Dry Docks

A basin, or *graving*, dry dock is simply an excavation or depression in the earth with one end in free communication with the sea. For dry-dockings, the seaward end is sealed off with a gate and the basin is dewatered.

One of the earliest graving docks was constructed in the late 1400s at Portsmouth, England, but it was not until the middle to late 1600s that the basin dry dock came into prominence.

Some of the earliest basin dry docks were simple excavations lined with timber that had a brick or concrete floor and fitted with a gate bearing against a sill to exclude the tide. The vessels entered at high tide, and the entrance gate was closed. As the tide receded, the dock emptied itself through a tidal sluice, and the vessel settled on blocks that had been prepared for it. At locations with little or no tide, the water had to be pumped out of the basin. As ships became larger, dry docks became important civil works. Solid masonry was adopted for lining the walls, and large pumps were added for rapid dewatering of the dock. A history of basin dry docks and further description of some of the early designs can be found in Cornick (1958), Colson (1894), and Hepburn (2003).

A present-day basin dock, when compared to its predecessors, differs significantly in terms of dimensions, materials, details of construction, and sophistication of the equipment, but the basic principles of operation and design are still the same. The general design of basin dry docks is covered in Mazurkiewicz (1980), PIANC (1988), DOD (2002), and BSI (1988).

General Features and Characteristics

The basic features of a basin dry dock, shown in Fig. 10-3, include the floor, sidewalls, head wall, and dock gate. There are three basic types of basin dry dock structures:

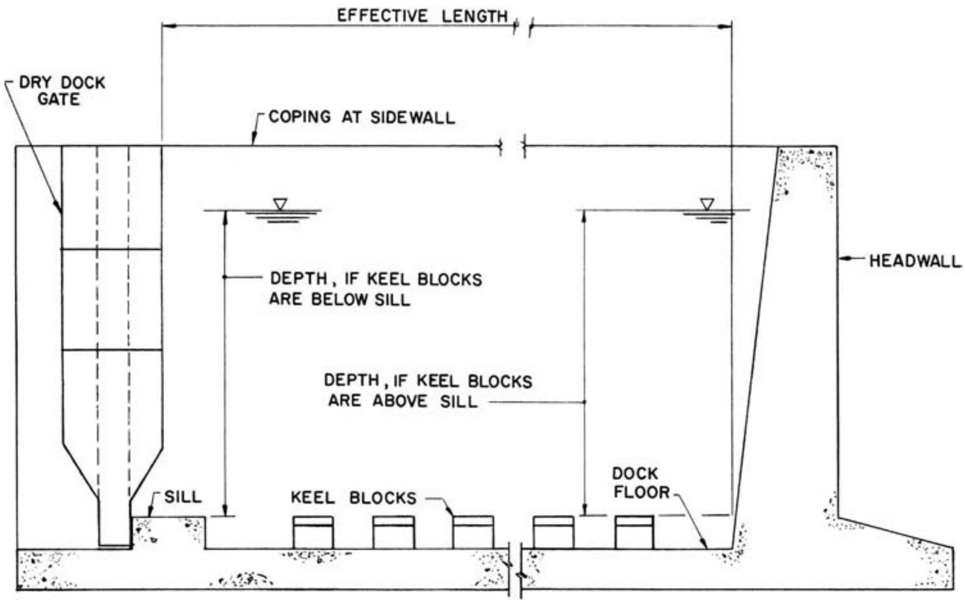


Fig. 10-3. Basic features of a basin dry dock

mass gravity, ground anchored, and underdrain (or pressure relieved). Various methods of construction use underdrains and massive stone or concrete slabs in combination with concrete or masonry retaining walls.

In general, basin dry docks are best suited for large vessels. Advances in materials and methods of construction have continually expanded the dimensional limits of basin dry docks. Shipbuilding and ship repair basins more than 400 m long, 60 m wide, and capable of handling vessels of all types and under all conditions, are common among recently completed docks. The construction of some of these large dry docks is discussed in Hassani (1973), Krauss and Plude (1972), Millard and Hassani (1971), and Tate (1961).

The operation of a basin dry dock is relatively simple. Once the ship and docking blocks have been properly prepared, the basin is filled with water until the water level inside the dock is equal to the level in the harbor. Then the gate is opened, and the ship is moved into the dock. Warping into the dock usually is accomplished by using head lines and spring lines on capstans or winches with assistance from harbor tugs. The dock gate is then closed, and the ship is properly positioned before the pumps are energized. Next, the water level is lowered, and the ship settles gently onto the blocks. During dewatering, the ship's position is carefully monitored, and the tension in the springing and breasting lines is adjusted accordingly. Undocking is similar, with the order of events reversed.

A vessel docked with a substantial trim relative to the keel block line can result in an extremely high load when making initial contact with the blocking, and the ship could become unstable or damage the blocks. Vessel trim can be accommodated to some degree by shoring or building the blocks on an incline, but as a general rule, an acceptable trim relative to the block line is 1 m for every 330 m of ship length. Beyond this limit, the ship's trim should be reduced or special preparations for blocking should be considered.

Dewatering a basin dock involves moving large quantities of water through large-capacity pumps. Basins are different from other types of docks in that energy consumption in basins during an operation is inversely proportional to vessel size; that is, the smaller the vessel docked, the greater the volume of water that must be pumped out, and the greater the amount of energy consumed.

Because the dock floor is far below yard level, natural light, air for ventilation, and access to the vessel are somewhat restricted. This situation tends to adversely affect working conditions; however, cranes traveling along the coping on each side of the dock or large gantry cranes spanning from sidewall to sidewall can easily move heavy equipment and large structural modules in and out of the basin.

Site Considerations

Site selection for dock construction involves careful evaluation of many factors, including topography, hydrography, geology, and meteorology. Because most of these factors have a direct effect on construction and operation of the dock, each must be carefully considered before a final site can be chosen. Locating the perfect site is extremely rare, so a series of trade-offs must be made between type and cost of construction, as well as operating and maintenance costs. Some preferences for dock location and chief concerns regarding construction are summarized below, but for a more detailed discussion on this subject see Stott (1957).

A basin should be located so that large ships can enter it without interfering with navigation. Normally, a dock is oriented perpendicular to the general shoreline. Providing a turning basin in front of the dock entrance is advisable to allow ships to align themselves with the axis of the dock before entering. At sites where there is a strong current parallel to shore, the dock may be oriented at a skew angle to the shoreline to avoid crosscurrents as a ship enters the dock. Similarly, orienting the dock parallel to the prevailing winds is desirable for maneuvering ships into dock. In general, a location in a protected harbor or within the confines of a breakwater is essential because ships should be sheltered from strong winds, waves, and currents during docking. Also, during the planning stages, a sheltered anchorage or standby berth should be considered in conjunction with the dock because there usually is some time between a ship's arrival in the yard and its going into dry dock.

Silting and scouring at the dock entrance and their effect on the gate and gate recess may warrant special consideration. The potential for excessive silting could indicate high future costs for maintenance dredging.

Tides and waves at the site have a direct effect on construction because they influence the elevation at the top of the gate and along the coping. Soil types and their strength, as well as groundwater levels, greatly influence the kind of structure suitable for the site, the depth to which it can economically be built, and the type of construction methods that can be used. The groundwater level is the most critical factor in the structural design of the floor and walls.

The design of a basin dry dock involves a wide variety of soil engineering problems, as discussed in Section 8.8. For this reason, a thorough subsurface exploration program must be implemented before the design of a new dry dock.

Finally, supporting shop facilities should be in close proximity. The availability of a large amount of lay-down space around the dock is a definite advantage, and some type of crane service is essential for moving material into and out of the basin.

Dimensions of a Basin Dry Dock

The key dimensions of length, width, and depth depend on the type of ships to be docked and whether the basin is to be used strictly for new construction, vessel repair, or a combination of both. The effective length of a basin is the minimum horizontal distance measured along the centerline between the head wall and the dock gate. This is illustrated in Fig. 10-3. The effective length should be at least 3 to 5 m longer than the overall length of the maximum design ship. To allow for propeller and shaft work, an additional 8 to 30 m should be provided. Docks that are extremely long can be partitioned off by an inner gate (Fig. 10-4) or gates, which provide operational flexibility and savings on pumping costs in docking smaller vessels. It is quite conceivable for one, two, or possibly more small vessels to occupy the dock simultaneously.

As shown in Fig. 10-5, the entrance width is the clear distance between the permanent dock fenders or wall structure at the dock entrance. The width at the entrance should be at least 2 to 3 m wider than the widest ship to be docked. The clear width within the basin normally is greater than the entrance width to allow space for equipment working on the vessel. Most basins built today have an effective length-to-width ratio of between 5:1 and 7:1, in approximate proportion to large contemporary vessels (Mazurkiewicz 1980). The inside width is established from the beam of the design vessel plus an allowance for clearance on each side of the vessel for working space. Fig. 10-6 shows a cross section with minimum clearances noted.

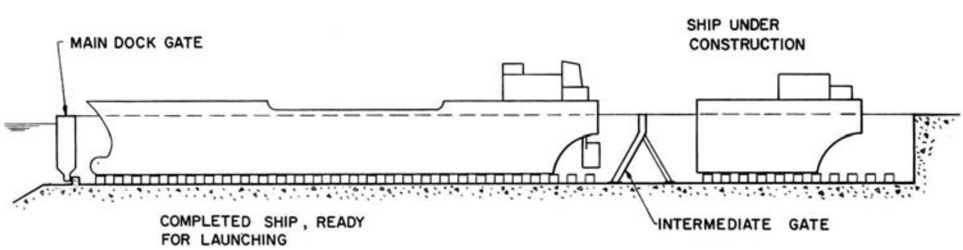


Fig. 10-4. Shipbuilding dock with intermediate gate

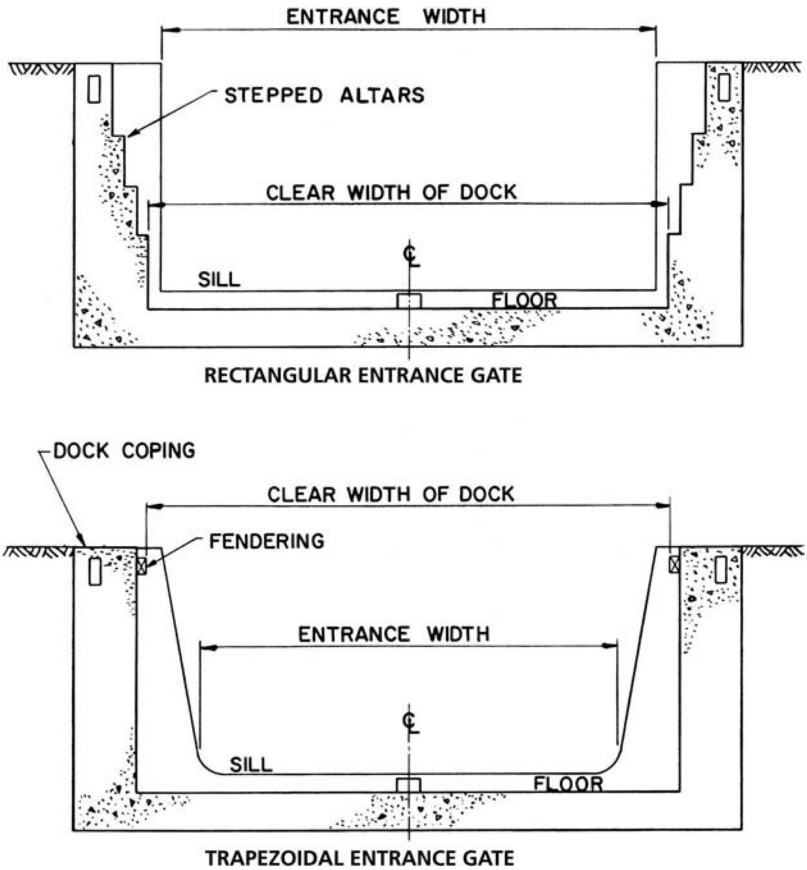


Fig. 10-5. Cross sections showing entrance widths

The required depth at the entrance should be decided only after careful study of all available water level data in conjunction with the vessel's drafts. Ships needing repair or routine maintenance, or those coming into dock in a damaged condition, require a deeper dock than vessels under construction, which generally are moved out of the basin once the main hull is watertight and before final outfitting is complete. Where appropriate, the designer should include an allowance for ship-hull appendages such as sonar domes or propellers, which sometimes extend below the vessel's keel. In areas where tide is minimal, the mean low water level (MLW) should be used in determining the depth of the entrance. Where docking or undocking delays cannot be tolerated while waiting for certain tide levels, as may be the case with warships, then the mean lower low water level (MLLW) should be used in determining the entrance depth. For commercial vessels, dry docks in tidal harbors may rely on high tide for docking and undocking operations. In all cases, the depth over the sill must include at least a 1-ft allowance for clearance.

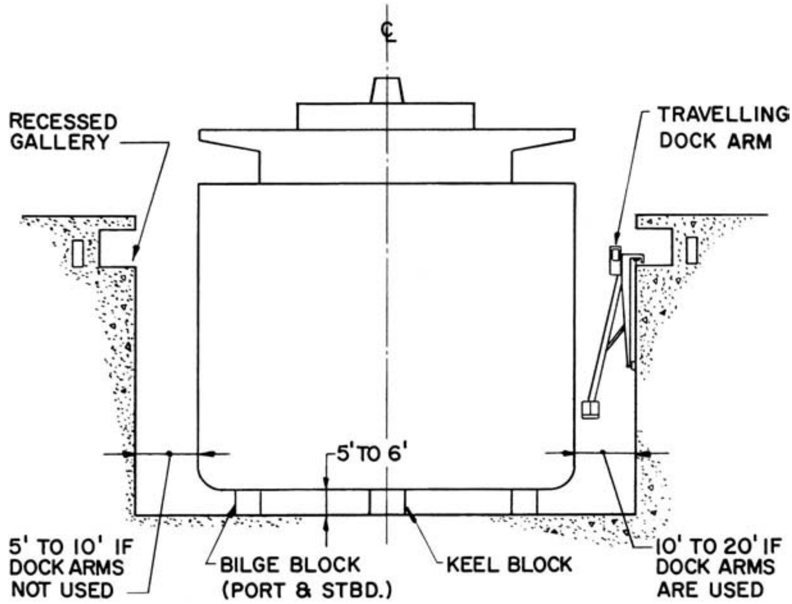


Fig. 10-6. Minimum clearances for construction or repair work

The depth to the floor of the basin usually exceeds the depth to the sill by a minimum distance equal to the height of the keel blocks (Fig. 10-3). At the head wall, the depth to the basin floor may be slightly less than at the sill if the dock floor is built on an inclination.

The depth of a basin has the single greatest effect on overall project cost. Extending the depth of the dry dock considerably increases construction costs, in proportion to the third power of the increase in depth (e.g., doubling the depth increases construction cost by a factor of eight) (Mazurkiewicz 1980).

Types of Basin Dry Docks

Each of the three basic types of basin dry docks—mass gravity, anchored, and underdrain (or pressure relieved)—relies on a different method to counteract the effects of buoyancy caused by the surrounding groundwater.

In a mass gravity design, the structure has sufficient mass to overcome the upward pressure of the groundwater acting on the underside of the floor when the dock is empty. Gravity docks can be constructed as monolithic frames or with floor slabs separate from the sidewalls. They also can be built with heavy floor slabs, which have their own weight as sufficient to resist the uplift. In docks where the floor slab is separate from the walls, the floor usually is made as a homogeneous slab or inverted arch supported by the walls. In this case, the walls can be made of steel sheet pile that resists pullout by virtue of soil friction, or they can be the reinforced-concrete

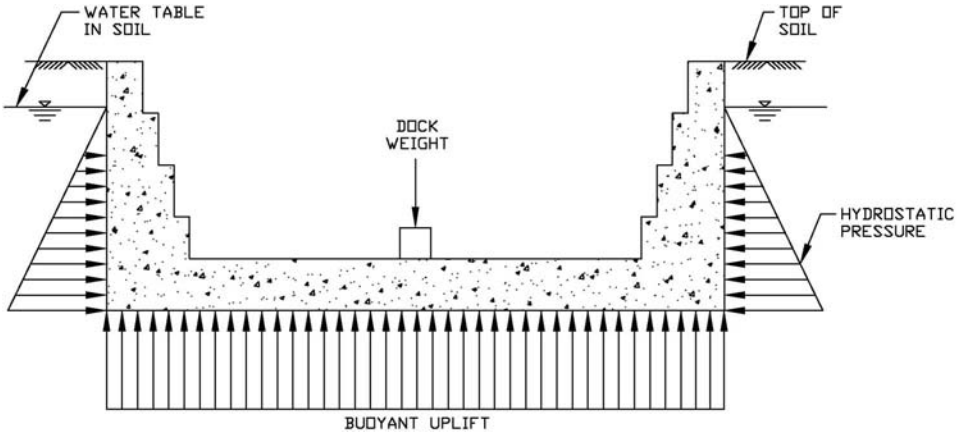


Fig. 10-7. Mass gravity dry dock with hydrostatic forces shown

retaining type of wall. In the latter case, the thickness of the slab is reduced under the walls, thereby taking advantage of the walls' weight. A mass gravity-type dock with hydrostatic forces shown is illustrated in Fig. 10-7.

The mass gravity type of dock generally is preferred for docks of significant width; but for docks of great depth, difficulties arise during attempts to lower the groundwater if the slab is to be cast under dry conditions. Moreover, with an increase in depth, the thickness of the floor slab also increases considerably. For docks of great depth, use of the gravity-type dock becomes labor-intensive and uneconomical.

In an anchored dock design, the hydrostatic uplift is opposed by the permanent weight of the structure plus some type of anchorage. The soil conditions must be such that they can provide bearing capacity for the floor slab loads and allow for the development of tension in pilings and/or anchors embedded in the soil. Where pilings are used, they may play a dual role, acting in tension when the dock is empty and in compression when there is a ship in dock. To avoid some of the disadvantages of a mass gravity design, the anchored type of dock relies on a relatively light floor slab. This type is less expensive to build and can be constructed in a shorter period of time, compared to a gravity dock.

In the design of gravity- and anchored-type docks, special consideration must be given to ensuring an adequate safety margin against uplift. According to PIANC (1988), a factor of safety of 1.1 is recommended for comparing the downward weight of the dock to the uplift forces. For this calculation, it is recommended that the designer assume the highest possible level for the groundwater and the lowest expected density for concrete with no ship or other loading on the dock floor. In anchored docks, the hydrostatic load usually is increased by a factor of 1.3 for the design of the anchoring system.

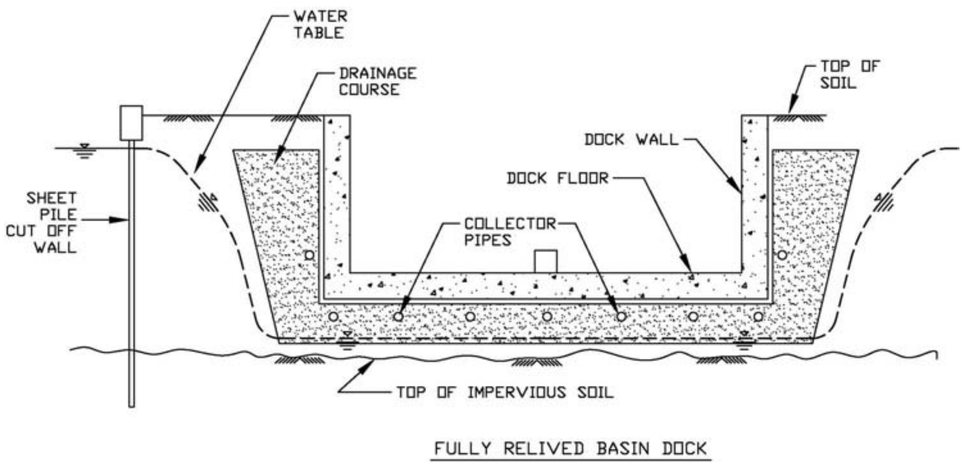


Fig. 10-8. Fully relieved mass gravity dry dock

In an underdrain or pressure-relieved design, the groundwater around the dock is lowered sufficiently so that no uplift pressure develops. A precise knowledge of the soil parameters is important. Data on the continuity of the impervious layer and its exact depth, as well as horizontal and vertical permeability coefficients, must be determined. A pressure-relieved dock may be fully drained, partially drained, or situated on impervious soil. A cross section of a fully relieved basin dock is shown in Fig. 10-8.

In docks where the pressure is relieved at the floor slab and the walls, an external drainage system is installed, which allows the groundwater level to be lowered. The drainage system is a series of pipes and culverts connected to the dock's pumping station. In docks where the pressure is relieved under the floor slab only, water flow to the drainage system is controlled by a sheet-pile cutoff wall installed below the dock walls; the cutoff wall extends to the impervious layer. This type of dry dock is illustrated in Fig. 10-9. Piezometers are recommended for fully or partially relieved dry docks to ensure that pressure relief is being maintained.

Also, depending on soil conditions, lowering of groundwater levels may be accomplished with a shallow dewatering system such as sand drains, a deep pressure-relief system such as deep wells, or a combination of both. An application of these methods for a basin dry dock at the Long Beach Naval Shipyard is described in Morrison et al. (1989). Basically, the system was designed to induce drainage from the backfill soils behind the dry dock walls through sand drains, into the underlying aquifer. The water subsequently was pumped from the deep wells to relieve hydrostatic pressure on the base of the dry dock. Some ground conditions do not permit lowering of the water table because this lowering can lead to consolidation of clays and silts and result in unacceptable settlements of surrounding structures.

Variations on methods of construction for the three basic types of docks can be found in Mazurkiewicz (1980) and PIANC (1988).

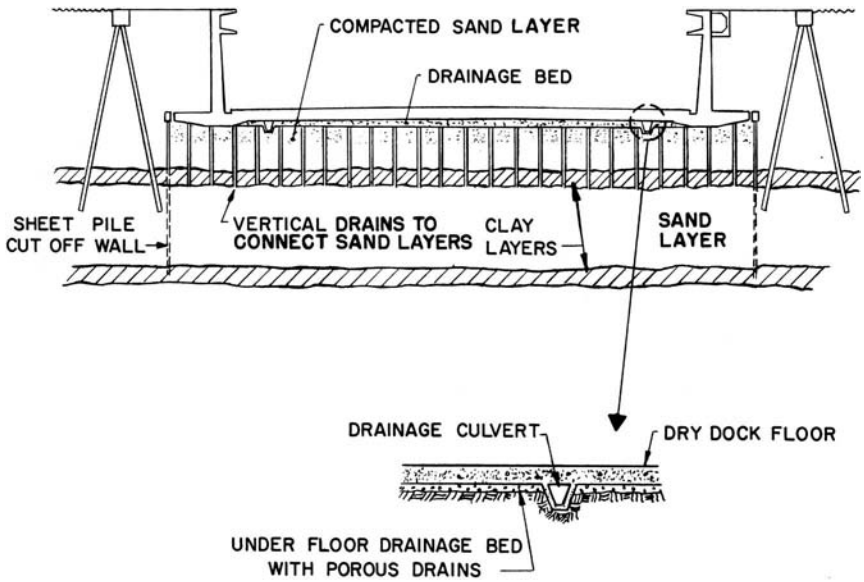


Fig. 10-9. Dry dock with underdrain system

Dock Floors

Today, practically all dry dock floors are built of reinforced concrete, but before the advent of reinforced concrete, brick arch and stone block floors were provided, both for structural strength and for mass. The floor slab is designed for several types of loading conditions, including the concentrated loading from a ship and, depending on the type of dock, the full hydrostatic uplift on the underside of the floor. The two main means of supporting the floor loads are direct bearing on the ground and end-bearing or friction-type pilings. Direct bearing on the ground requires the slab to have sufficient strength to spread the load to the soil, and the soil to have adequate bearing capacity to support the load without excessive deformation or settlement. The analysis is similar to that for a beam on a continuous elastic foundation.

Dock floors may be built as horizontal or with an inclination. An inclined floor can slope longitudinally toward the dock entrance or transversely, from the centerline to the sidewalls. The floor generally slopes at an inclination of about 1:300, which enhances drainage of the dock during dewatering.

In docks where the floor is horizontal, better drainage is achieved by inclined dewatering channels or troughs recessed into the slab. Transverse dewatering channels are oriented perpendicular to the axis of the dock, directing water flow toward the sidewalls, where channels run longitudinally, allowing water to flow to sumps and into the main intake culverts.

Control joints may be required in the floor for shrinkage. Expansion joints rarely are required, as the temperature of the slab remains fairly constant because it is in contact with the underlying groundwater.

Dock Sidewalls

Originally, docks were designed with sloping walls that had many offsets called altars, as shown in Fig. 10-5. Today, most docks have vertical walls. The sidewalls are designed for a variety of loadings, including earth pressure and surcharge, groundwater pressure, water pressure from inside the dock, and loadings from equipment and fittings such as cranes, traveling dock arms, ship-hauling gear, bollards, and services. In addition to these loadings, the walls must be designed for the hydrostatic loads transmitted up through the dock floor. In many locations, earthquake loads must also be considered.

The main types of dock wall construction include mass concrete, reinforced concrete, sheet piling, and caissons. The use of mass concrete was prevalent during the early twentieth century and may still be economical in some instances, but it generally is not common in modern construction. Many of the docks built within the past 40 to 50 years were built with reinforced concrete walls.

When sheet-pile walls are used, slightly heavier sheeting than that required by design is recommended to allow for corrosion. Special considerations are necessary if the walls are to carry vertical loads from cranes and building columns; the watertight integrity of the installed sheeting at the joints is also of concern. If a groundwater cutoff wall is required, then sheet piling can be used to great advantage by terminating the sheeting in an impervious layer some distance below the dock floor. This construction effectively reduces the amount of water to be pumped from an underdrain system.

In some cases where real estate is at a premium, it may be possible to extend the basin out into the channel by using interlocking floating caissons for the walls. Once in place, the caissons are filled with earth to secure them.

The three basic wall profiles are illustrated in Fig. 10-10. The uniform vertical profile requires that all equipment, piping runs, electrical raceways, and so on, be set

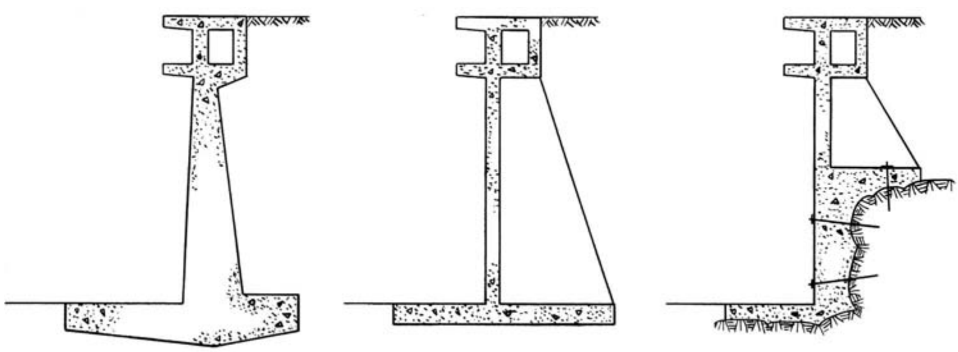


Fig. 10-10. Typical wall profiles

inside the closed gallery with all service outlets located on the dock coping. The vertical profile with recessed or cantilever galleries just below the coping provides a place for service lines and service outlets. In all cases, the walls may be inclined as well as vertical, and the wall profile should be such that it makes total use of the full floor width. The layout of the dock wall should be closely coordinated with the many services and other equipment required along the coping.

Structural Analysis of Basin Docks

The structural analysis of a basin dock depends on the type of dock being constructed. In general, the following loading cases need to be considered:

- Dock under construction,
- Dock empty (maximum hydrostatic uplift),
- Maximum ship load (minimum hydrostatic uplift),
- Dock full of water, and
- Super flood condition (when applicable).

A full hydrostatic dock needs the strength to resist the maximum hydrostatic forces that develop on the walls, floor, and gate. The walls must resist the additional side loading from the soils. A fully relieved dock does not have the hydrostatic loads because this pressure is relieved by draining the groundwater.

All types of docks must support the weight of the ship as it is supported by the blocking system. Usually it is assumed that the heaviest load occurs along the dock centerline where the keel blocks are normally positioned. It is best to design the floor slab such that the fully loaded keel block can be placed anywhere along this line. The side blocks usually support a smaller percent of the load, but this amount varies depending on the type of vessel being dry docked. Some floor slabs are designed to place a block anywhere, whereas others require the side block to be located in specific zones on the floor (such as over lines of piles). The more flexible the position of the side blocks can be, the better chance of accommodating a wide variety of docking block layouts.

The floor slab should also be designed for vehicle loads such as trucks, forklifts, manlifts, cranes, and self-propelled modular transporters (SPMTs).

The finite-element method of analysis is the preferred method for analyzing the complex structure of a basin dry dock. Whereas previous analyses were accomplished by using strength-of-material concepts and simplified theories of elasticity, this method considers the interaction between the soil and the structure, as well as the stress–strain characteristics of the structure itself. A detailed description is beyond the scope of this text, but the reader can consult Wu et al. (1988), which describes the method and includes examples of the results of a typical analysis. Such an analysis provides a thorough evaluation of the safety of a dry dock subjected to static soil and hydrostatic pressures, ship loadings, and earthquake-induced dynamic loading.

Pumping and Flooding Systems

Pumping and flooding systems control the emptying and filling of the dry dock, which must be accomplished in a gradual, controlled manner. The dewatering process must allow for the ship to ease down onto the blocks, and the flooding must be controlled to the point where there is no large inrush of water, which could disturb the blocks.

Pumping and flooding times vary from dock to dock, but the times usually are independent of dock size. Normal pumping times for a basin dry dock are in the range of 2 to 3 hours, but some docks require up to 12 hours or more for dewatering. Normal flooding times, on the other hand, are about half as long. In general, for dry docks engaged primarily in repair work, the less time required to pump and/or flood the dock, the better. This translates to shorter docking times and thereby decreases vessel turnaround time and idle time for shipyard workers. For basins in which the emphasis is on new construction, longer pumping and flooding times are acceptable because the dock is operated less frequently. Computation of flooding and pumping times is described in DOD (2002). The basics of the dewatering and flooding systems are illustrated in Figs. 10-11 and 10-12, respectively.

The three main types of dewatering pumps used in basin dry docks are centrifugal, mixed-flow, and axial-flow pumps. Centrifugal pumps may be installed with either a horizontal or a vertical shaft, but the mixed- and axial-flow pumps are vertical installations. Pump types and guidelines for proper pump selection are described in detail in Grieve (1960) and Wauchope (1958). Final pump selection must take into account dewatering time, power consumption, initial cost, and operating cost. The number of main pumps usually ranges from two to five. A single pump never should be used because a breakdown would render the dock inoperable.

Dry dock flooding systems use several types of valves; the most common are sluice valves, which are used in the flooding culverts; equilibrium valves, for filling the dock; and gate valves and butterfly valves, which can be used as isolating valves and flood valves in the gate. Sluice valves, found mostly in small, older docks, can vary in shape and in materials of construction. Their shape is determined by the shape of the culvert; the materials of construction can include timber, cast iron, or steel. Equilibrium, or cylindrical, valves consist of a large vertical cylinder supported in vertical guides. The valves are located in compartments with direct access to the sea, and the valve bottom consists of a seal set over the flooding culvert (Fig. 10-12). When the valve is lifted, water flows radially into the culvert. Because the cylinder is extremely large, its weight is counterbalanced so that a relatively small force is required to open the valve. Isolation valves in the suction and discharge culverts provide a means for repairing or performing maintenance on a pump without affecting the rest of the dewatering system. Valve operators for the larger valves are electric, hydraulic, or compressed air. All should have a provision for manual

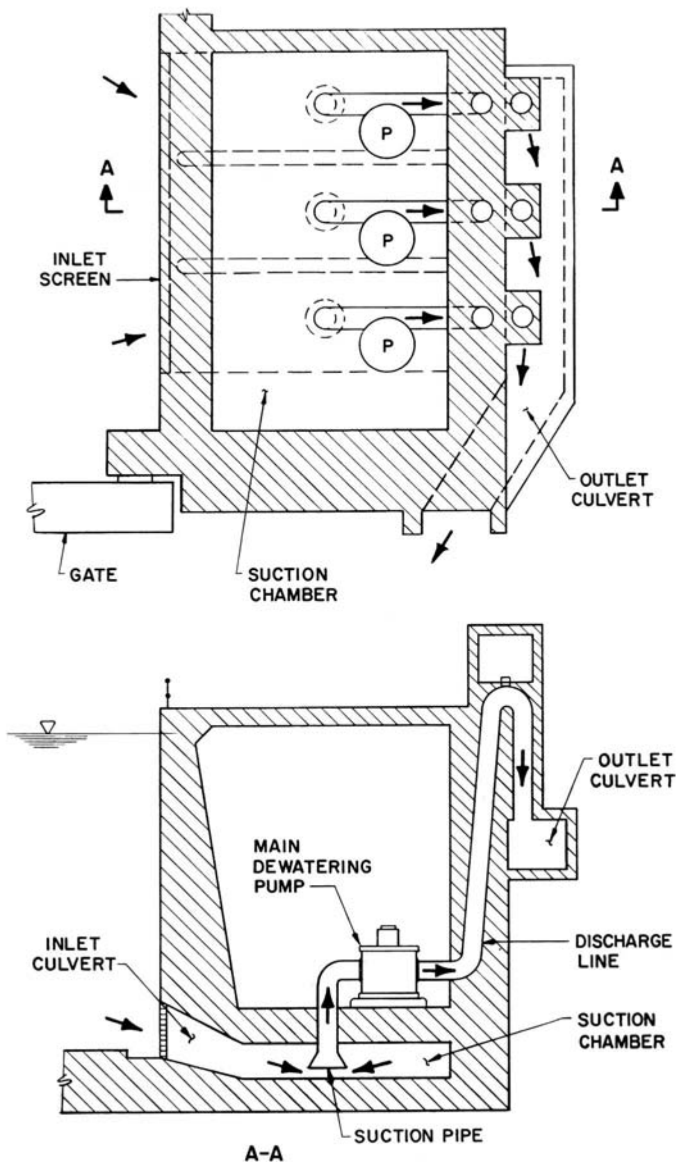


Fig. 10-11. Dry dock dewatering system

operation in the event of system failure. More information on culvert sluices and valves can be found in DOD (2002) and Gardener (1958).

Any dry dock that is certified by the U.S. Navy to dock naval vessels must be protected from sources of potential flooding, such as flooding inlets or backflow through pumps, by two methods of protection. Combinations of valves, sluice gates, and stop logs may be used to meet this requirement (DOD 2002).

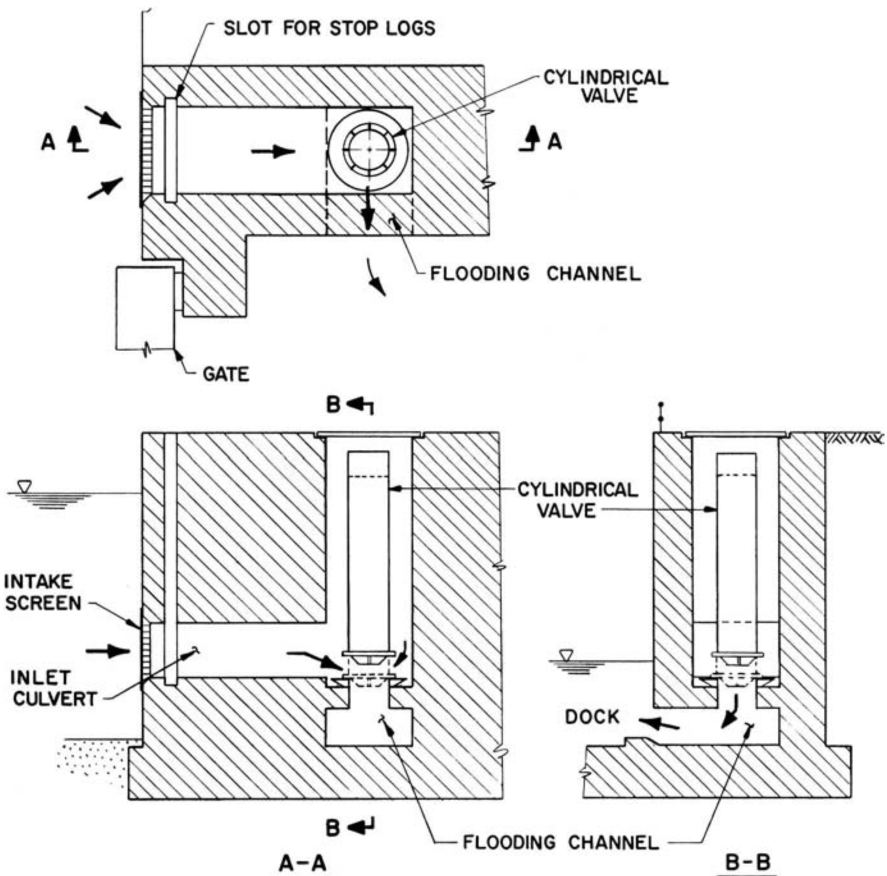


Fig. 10-12. Dry dock flooding system

A separate building or chamber called a pumping station, usually built into the wall of the dock, contains the dewatering pumps and valve operators, as well as other smaller pumps and equipment for discharging rainwater and providing firefighting or ballast water, or, in the case of an underdrain-type dock, pumps for the discharge of groundwater. A typical pumping station is illustrated in Fig. 10-13. Local control of the pumps and valves is provided in the pumping station. In addition, remote operation from a central control house at yard level is found at most facilities.

The preferred location for the pumping station is within one of the sidewalls near the entrance to the dock to minimize the lengths of suction and flooding culverts. The dock's sidewall requires special detailing for the pump house. In the case of parallel twin docks, the ideal location for the pumping station is in the wall between the docks so that the same pumping station can be used to dewater each dock. For docks fitted with an inner gate, it is preferable to have the pumping station near the inner gate sill or recess to serve both sections of the dock efficiently.

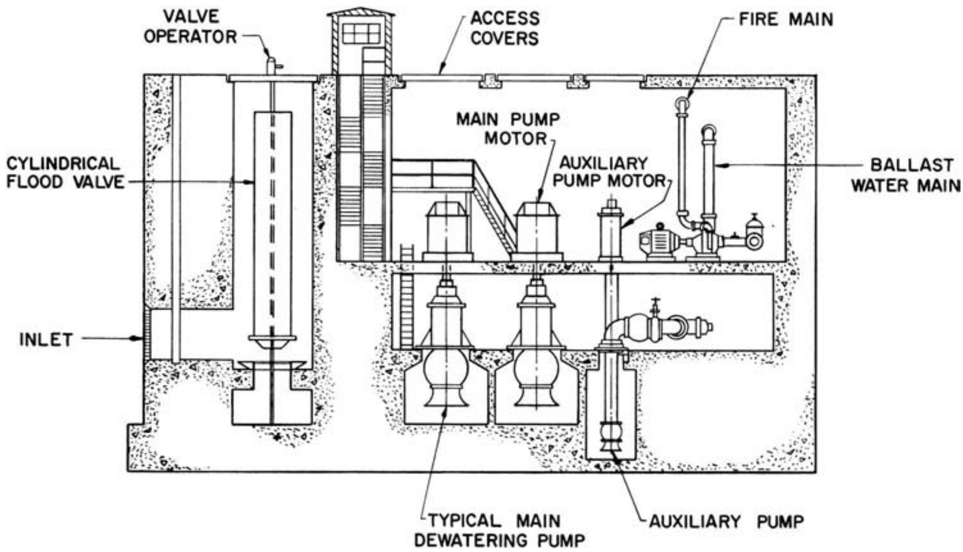


Fig. 10-13. Typical dry dock pumping station

The arrangement and size of the pumping station depends mainly on the type of pumps selected. If horizontal-shaft pumps are chosen, they must be mounted as close to the floor level as possible because of their limited suction height. The pump motor is coupled to a shaft extension from the pump and is mounted on the same level as the pump. This arrangement requires a large floor area for each pump/motor unit.

The most recent docks have used vertical-shaft pumps. The drive motor is on a common vertical shaft with the pump and is mounted directly above it. This arrangement has the advantage that it significantly reduces the amount of floor space required, resulting in considerable construction savings (Fig. 10-13).

In a modern pumping station, it is necessary to have a reliable source of electrical power, sufficient lighting, a communication system consisting of a telephone or radio and loudspeaker system, and adequate ventilation and heating. Also, if there are auxiliary pumps in continuous operation for handling groundwater seepage, it is imperative that an independent power source be available in the event of a power failure.

All pumping and flooding operations are controlled from the control center. The center is usually in a special tower or building conveniently located with respect to the basin. In addition to the pump and valve controls, there are gauges for monitoring water levels in the dry dock as well as the major suction chambers. The control center has electrical status boards and indicators; an alarm system for fire, equipment failure, main power source failure, and so on; a communication system; fire pump controls; and controls for all auxiliary pumps. The control console usually has a mimic diagram showing schematically the arrangements of culverts, sumps, pumps, and valves as well as their open/close or run/non-run status.

Some docks are designed to allow superflooding. These docks have the capability to pump water into the basin, raising the level in the basin above the level of the water outside of the dock. Superflooding increases the capability of the dock by allowing deeper draft vessels to be dry docked. To superflood, the dock requires pumping capability to allow pumping water into the dock in addition to the normal dewatering capabilities. It also requires an outboard-facing caisson seat to resist hydrostatic pressures on the gate that are directed outboard because of the higher level of the interior water. This is the opposite direction of hydrostatic forces that a gate normally encounters.

Dry Dock Gates

The dry dock gate provides the necessary separation between the basin and the harbor. These gates vary widely in terms of design, configuration, and means of operation. There are five basic types of gates: floating, hinge, sliding, mitre, and flap gates. Examples for each type are illustrated in Figs. 10-14 through 10-18. The type, shape, and overall dimensions are influenced by a variety of factors:

- Width of the entrance;
- Depth of the entrance;
- The head of water to be retained during various phases of operation;
- The reverse head of water in docks capable of being superflooded;
- Tides, winds, and waves as they affect the gate's height;
- The speed of operation. For repair docks, a gate operating time of 10 to 15 min is preferred; for building basins, an acceptable time is about 30 min;
- Cost of construction, including associated civil works and operating mechanisms;
- Depth available below the sill just outside the dock entrance, and the possibility of siltation;
- Availability of parking space for the gate against a pier or quay wall;
- Cost and ease of maintenance;
- Labor and mechanisms required to operate the gate;
- Power source required to operate the gate;
- Access across the top of the gate for pedestrians and/or vehicles;
- On-site versus off-site location for construction of the gate; and
- Special considerations, such as ice loads.

Whatever the type of gate chosen, it is imperative that it make a tight seal against the dock entrance. Hard rubber is the most common materials used for gate seals. The seal must be accurately shaped to provide a good fit; its cushioning ability makes up for irregularities and unevenness along the face of the concrete.

Buoyancy is an important consideration for all gates because they should have sufficient buoyancy to aid their removal from and placement in the sill.

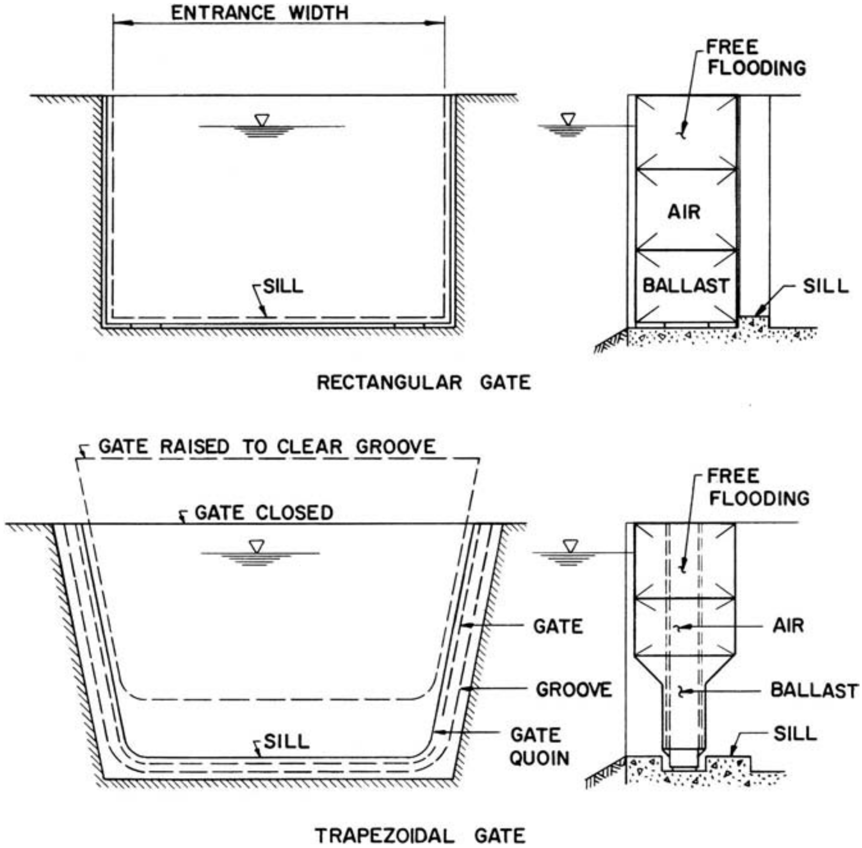


Fig. 10-14. Typical free-floating gates

Floating Gates

Free-floating gates are the most widely used. They are usually constructed of steel, but they can also be constructed of reinforced concrete or prestressed concrete. A floating gate is completely independent of the dry dock. Once it is free of the sill, it is towed to a pier or quay for mooring, out of the way of ship traffic. A floating gate is particularly well suited for wider docks. The top deck can be designed to allow heavy vehicular traffic to pass over it. Floating gates are usually made reversible (i.e., the gate can be installed such that either side is facing into the dock). This arrangement allows maintenance on either side of the gate. Floating gates can be constructed at a remote location and towed into position for immediate use. Their main disadvantages are that the operating time is longer than that of flap-type gates, and they require a larger crew to operate pumps, valves, winches, and capstans and to handle lines during gate removal and placement.

Floating gates can have a variety of cross-sectional shapes; and in elevation, the sides may be vertical or inclined, as shown in Fig. 10-14. Floating gates have a top deck

Fig. 10-15. Floating caisson gate under construction

called the *weather deck*. This is where the cleats, chocks, and capstan are located for use when removing and storing the gate. Below the weather deck is the machinery deck. This is a watertight deck where the pumps, valve operators, and control panel are located. The ballast tanks are located below the machinery deck. The machinery deck is positioned vertically to limit the amount of ballast that can enter the tanks. This arrangement ensures that the gate cannot be inadvertently submerged should the flood valves be left open. The ballast tanks are subdivided for stability and ballast control. Fig. 10-15 shows a typical floating caisson gate under construction.

A floating gate usually has minimal stability when it is deballasted. Extreme care must be taken when designing and building the gate to ensure that the desired total weight and vertical center of gravity are not exceeded because these will have a profound effect on stability. As the amount of water ballast changes, the center of gravity of the gate shifts, as does its metacentric height (GM) (see Section 9.2). The designer must ensure that the gate remains stable through all levels of ballast. A minimum 1.0-ft GM should be strived for. To improve stability, most floating gates contain heavy-weight concrete ballast near the bottom. The stability of floating gates is discussed in more detail in Hassani (1987).

Hinge Gates

Hinge gates are modified versions of floating gates, made with a vertical hinge on one side. The hydrostatic principles of the hinge gate are similar to those of the floating gate. The gate is opened by means of wire ropes and winches. An alternative method is to connect a boom operated by a hydraulic ram capable of pushing or pulling the gate. The hinge gate has many of the same advantages and disadvantages

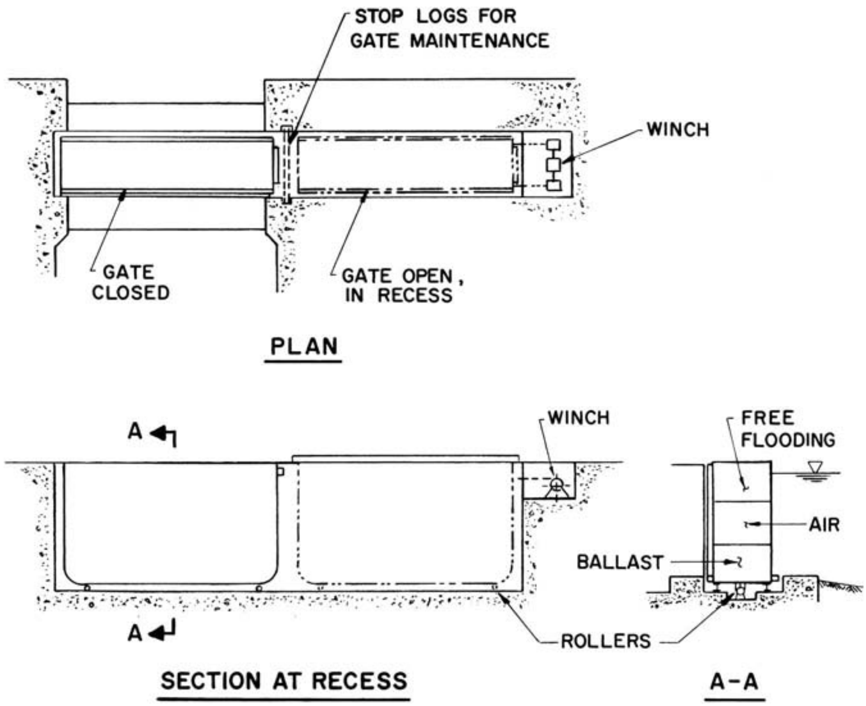


Fig. 10-16. Sliding caisson gate

of the floating gate. For maintenance, the gate can be disconnected from the hinges and moved to another location for topside repairs or dry-docking.

Sliding Gates

Sliding gates typically are floating pontoons or caissons made of steel or concrete. When the dock is opened to the sea, the sliding gate is shifted transversely into a special pocket or recess built into the sidewall of the dock, as shown in Fig. 10-16. Sliding gates are similar in construction to floating gates, and they can be used for outer as well as intermediate gates. They are easy to maneuver, requiring only a small crew, and are designed for fast opening and closing. Their main disadvantage is the need for a special storage chamber built into the sidewall of the dock; such a chamber can add significantly to the overall cost of construction. The usual configuration is rectangular, but some are built in an arched shape. The typical width-to-height range is 1:4.5 to 1:6.5, in order to allow for traffic on top of the gate (Mazurkiewicz 1980).

The gate slides on a system of wheels or rollers on rails and also can slide on a smooth surface. In either case, the gate is deballasted to minimize the reaction on the sliding tracks. Of these designs, those without rails provide better watertightness, as the entire bottom of the gate presses against the sliding surface to form a tight seal. The gate can be built with meeting faces on both sides if there is the possibility of reverse head pressure. Despite the fact that the gate is deballasted before opening, considerable breakaway force is required, especially if the gate has been sitting for a long time. Wire cables connected to electric or hydraulic winches normally are used for moving the gate, although hydraulic rams sometimes are installed to provide the initial breakaway force.

Mitre Gates

The mitre gate is split at the middle and consists of two leaves, each rotating about a vertical hinge at the sidewall of the dock. The vertical axis is supported by an upper and a lower bearing, and the entire hinge assembly is recessed into the wall of the basin. The leaves are shaped so that they form an arch across the dock entrance to resist water pressure. Watertightness is achieved by mating pieces of greenheart timber or hard rubber on the mitre posts at the centerline, as well as timber or rubber at the wall hinges and along the bottom at the sill. A mitre gate is fast-operating, requiring only a small crew, and the gates are designed with some buoyancy to minimize the load on the hinges and eliminate the need for bottom rollers.

Upon opening, the gate leaves enter recesses formed in the sidewalls at the dock head. Thus, the entrance width can be as wide as the main dock chamber, as shown in Fig. 10-17.

Some disadvantages include the gate's inability to accommodate reverse loading or roadway traffic across the top. Also maintenance of the structure is difficult, and it is generally considered unsuitable for large entrance widths.

Flap Gates

A flap gate consists of a single leaf spanning the entire width of the dock, hinged horizontally on two heavy trunnion bearings at the level of the dock floor. Operation of the gate is simple and speedy, as it can be raised or lowered in about 5 min by electrically driven winches. When open, the entire gate is below the sill level of the dock, thus posing no limitation on the water depth at the sill. It can be accommodated in the waterway or channel adjoining the dry dock and does not require a special recess in the floor or wall structure. The gate is designed to be semibuoyant to reduce the load on the wire rope used for opening and closing the flap. The design of a flap gate is greatly influenced by the entrance height-to-width ratio. The gate may be designed with a deep box girder forming the top portion of the gate spanning the entire entrance and with the lower portion a series of stiffeners spanning vertically between the sill and the girder. Alternatively, the gate may be designed as a series of heavy beams supported on three sides—the two sidewalls and the sill.

For wide entrances, the simple flap gate is too heavy and uneconomical. In this situation, a strutted flap gate may be considered. When closed, the gate is supported at intervals by a series of inclined struts designed to take the horizontal water

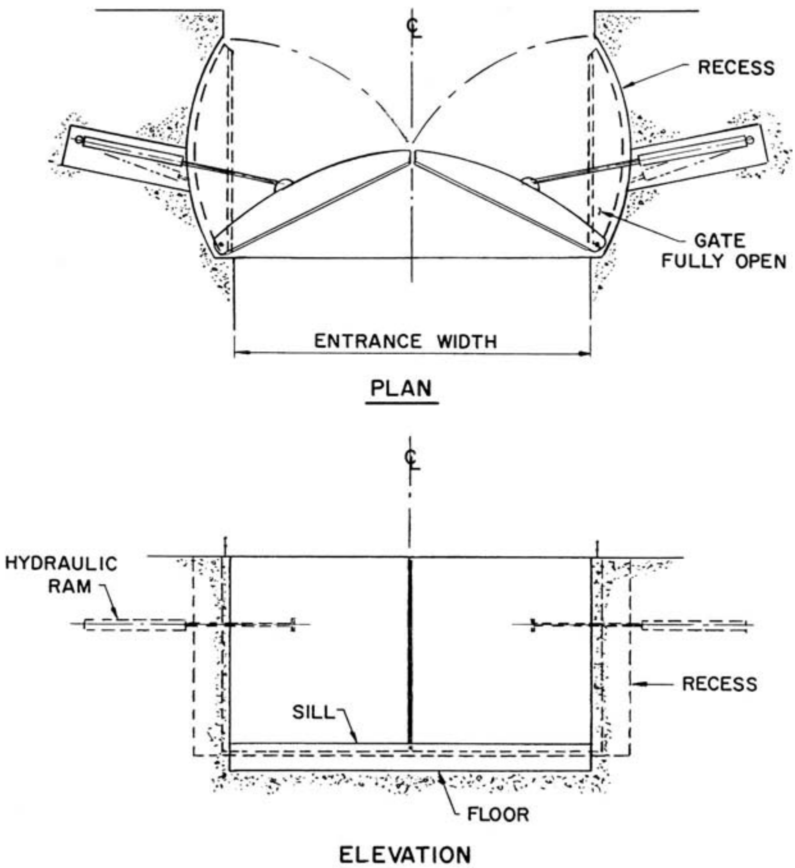


Fig. 10-17. Mitre gate

pressure. Another alternative is the cantilever flap gate. This gate is arranged to cantilever from the sill just above where it is hinged (Fig. 10-18). The tendency in dock construction has been to eliminate complex devices and to simplify the process of opening and closing the dock chamber. The flap gate has become the most economical and the most reliable type of dry dock closure. A more detailed treatment of the subject of dry dock gates can be found in Mazurkiewicz (1980), Hassani (1987), Walker (1958), Baily (1947), and Danks (1958).

10.3 Marine Railways

The marine railway dry dock is a mechanical means for lifting a ship out of the water to an elevation above the highest tides. Operating on the basic principle of the inclined plane, the marine railway comprises a cradle or carriage that is lowered into the water along a sloping track on a system of rollers or wheels, until the vessel to be

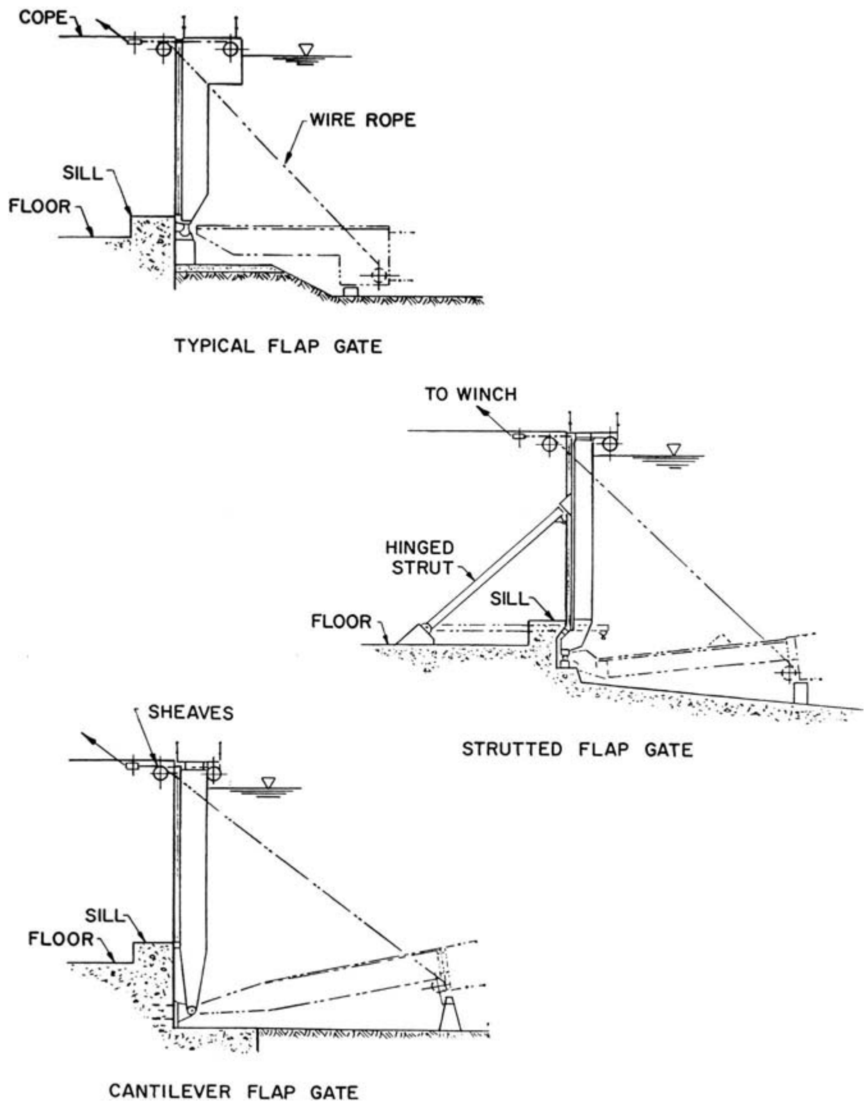


Fig. 10-18. Flap gates for basin dry docks

dry docked can be floated over it. Once the vessel has been centered, the cradle then is hauled up the track until the vessel grounds out on blocking. The operation is complete when the cradle deck is clear of the water, leaving the ship high and dry. The railway is designed so that the dry-docked vessel is positioned at or above the level of the yard, allowing materials and staging to be easily moved into position from the adjacent yard area. Also, because the cradle is virtually open on all sides, there is free circulation of air and good illumination, allowing vessels to be repaired quickly and efficiently. Fig. 10-19 shows the major components of a marine railway.

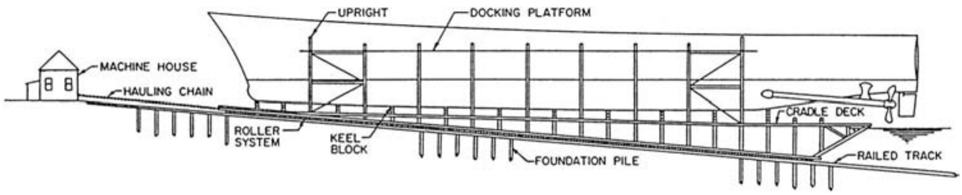


Fig. 10-19. The major components of a marine railway

The marine railway was developed during the mid-1800s, originating from the English-type slipway. The features introduced on the first marine railway in East Boston, Massachusetts, which differentiated it from the slipway, included higher keel blocks and a deck over the cradle, docking platforms for people to walk along while handling lines during a docking, and steam power instead of the power of people and horses.

A marine railway of modern design is fast-operating, reliable, and extremely durable in the harsh environment of the sea. In general, it is economical in both installation and operation when compared to other facilities in the same capacity range.

The marine railway is best suited for a capacity range of 100 to 4,000 tons. The type of ships to be docked and any special restrictions of the site usually dictate the final dimensions of the railway. Fig. 10-20 gives the usual dimensions for marine railways.

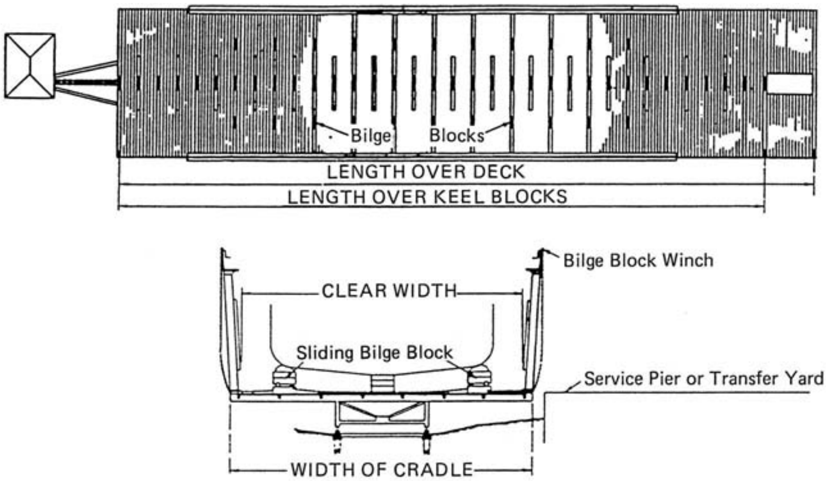
Marine railways can be of either end-haul or side-haul designs. Most modern railways are the end-haul type because that type of operation is generally safer, less complicated, and more economical to construct than the other. Also, the end-haul type requires only about one-third as much water frontage as the side-haul type.

Site Considerations

The marine railway is best suited to sites with gradually sloping bottoms. In areas where siltation is a problem, the natural bottom at the offshore end of the track should be below the track grade. A site in a sheltered harbor with natural protection from strong winds, waves, and currents is essential. For some locations, the formation of ice is a serious concern, as ice can severely hamper the operation of a marine railway. Because the trackage is fairly long, a sufficient area must be available to avoid infringement on the channel. Favorable subsurface conditions are essential for supporting the anticipated loadings. The capacity of the foundation determines the per-foot or lineal capacity of the railway.

Track and Foundation Design

The load that the ship exerts is transmitted directly to the track and foundation through the cradle and roller (or wheel) system. The design loadings for the track



LIFTING CAPACITY in tons of 2000 lbs.	LENGTH OVER KEEL BLOCKS in feet	LENGTH OVER DECK in feet	WIDTH OF CRADLE in feet	CLEAR WIDTH in feet	DEPTH FORWARD in feet	DEPTH AFT in feet
100	78	78	30	26	6	11
200	90	90	32	26	6	11
300	102	102	34	28	7	12
400	115	115	36	30	7	12
500	128	128	38	32	8	13
600	140	150	40	33	8	13
800	160	172	42	35	9	14
1000	180	195	44	37	9	14
1200	200	215	46	39	10	15
1500	220	235	50	42	11	16
2000	240	255	54	46	12	17
2500	270	285	56	48	12	17
3000	300	320	60	52	13	18
3500	320	340	65	57	13	18
4000	340	360	70	62	14	18
4500	360	380	72	63	14	18
5000	380	400	74	65	15	19
6000	400	420	76	67	16	20
7000	420	450	77	69	17	21
8000	440	470	78	72	18	22

NOTE:–
These dimensions may be modified to suit requirements.
The length of the track varies with the slope which conforms to the natural conditions of the site.

Fig. 10-20. Typical dimensions for marine railways

vary along the length of the track. For example, the inshore portion must be designed for the dead load of the cradle plus the live load of the ship. The offshore portion of the track is designed for the dead load of the submerged cradle and no live load. Moving inshore, the design load steadily increases from the point at which the vessel first grounds on the blocks until the vessel is completely out of the water, and the full live load and dead load are realized.

Depending on the natural shoreline, availability of space, river or tidal currents, variations in water level, and desired lifting capacity, the declivity of the track may

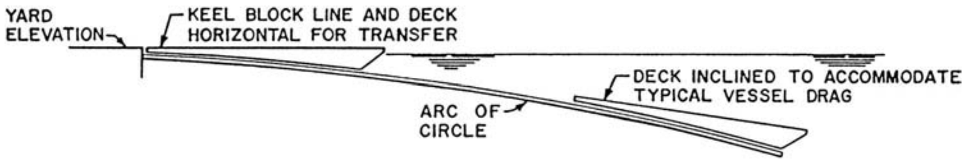


Fig. 10-21. Profile of a curved track

vary from 1 in 10 to 1 in 30. In most cases, the slope of the track is carefully selected to minimize or completely eliminate dredging.

The track must be constructed with a smooth gradient. A trend in railway design has been a departure from a straight gradient and development of a track constructed on a vertical circular curve (Fig. 10-21). This geometry causes the cradle to rotate as it travels along the track, making it possible for the keel blocks and cradle deck to be on a declivity when submerged, so as to be closely aligned with the trim or drag of a vessel. It also allows the cradle to be horizontal in the full-up position, which is necessary in the case of transfer to shore. A secondary benefit of the curved-track design is that it can provide necessary water depths while minimizing the overall track length. With respect to water depths, the optimum design provides for docking at mean low water, especially at locations where the tide range is small.

The earliest small-capacity marine railways used mud sills or sleeper ties laid on a bed of gravel. As load concentrations increased, the need for greater foundation capacity became apparent. Today, pilings of timber, steel, or concrete are used. When there is rock available close to the track, concrete footings can be used to support the track. The footings must be designed with the proper aspect ratio for stability and spaced to be compatible with the lineal load capacity of the railway.

Early track designs were made entirely of timber. Today a variety of materials are used, including steel, concrete, and timber. The most common track arrangements are composite structures of concrete and steel or concrete and timber. The composite design consists of a reinforced concrete section above the low-water line and steel or timber for the submerged portions. New steel track can be fabricated in the dry in sections approximately 12 to 20 m (40 to 66 ft) long. The sections then are floated into position and submerged where they are secured to steel piles. Compared to the all-timber track and foundation, this type of structure lends itself much better to heavy waterfront construction methods. The finished product is free from marine borer attack and, with good protective coating and cathodic protection systems, is durable in seawater.

The track structure may be composed of two, three, or four ways, depending on the vessel type and size. Most marine railways today are built with a two-way track arrangement.

Cradle Design and Construction

For durability and strength, the cradle superstructure on a modern railway is built of steel, including the transverse beams, runners, columns, and uprights that support the docking platforms. The cradle deck and blocking are made of wood. The cradle structure must have strength and stability to support the ship, and yet at the same time be flexible in longitudinal bending and in torsion to accommodate any irregularities that may occur in the track and/or the ship. Usually, the cradle runner structure is a simple running log for the inshore portion and a Vierendeel truss for the aft portion. The aft portion of a typical steel cradle under construction is shown in Fig. 10-22.

The gauge of the cradle runners must be wide enough to provide stability against overturning from wind, current, or seismic loads. As a general rule, the gauge of a railway is approximately one-half the beam of the widest vessel and/or one-third the width of the cradle.

The cradle platform is composed of a series of long and short transverse beams. The short beams are equal in length to the runner gauge and are designed to take loads from the keel blocks only. The long beams extend symmetrically beyond the cradle runners and are designed to support the bilge blocks as well as the keel blocks. Beams may be spaced at any increment, but the generally accepted pattern calls for long beams at 3.6 m (12 ft) on center with short beams in between at 1.2-m (4-ft) or 1.8-m (6-ft) spacing.

Fig. 10-22. Photograph of built-up cradle under construction

A divided cradle is one that is built in multiple sections, usually two, that can be operated as a single unit or detached for independent operation. Its design provides operational flexibility and facilitates cradle maintenance. The divided cradle allows for docking one large ship or two smaller ships. In the latter case, it is possible to have a ship docked on the forward section, for long-term repairs, while the aft section serves ships requiring minor repairs and maintenance. Maintenance of a divided cradle is somewhat easier than that of a one-piece cradle because it is a relatively simple matter to float away the bow cradle and then haul the after cradle completely clear of the water.

Vertical steel beams, called uprights, are fitted at the end of the long cradle beams. The uprights serve several functions. They support a continuous platform that is used by docking personnel for handling lines during the positioning of the ship. Winches that operate sliding bilge blocks usually are mounted on stands spaced along the docking platform. Also, the uprights are fitted with timber fendering and designed to resist a modest amount of load from ship impact during a docking operation.

Blocking Considerations

Railway dry docks typically are fitted with a standard all-timber blocking system that is 3 to 4 ft high. In laying out the block line on a cradle, it is advisable to have the top of the keel blocks parallel to the ship's keel when contact is made; otherwise, tipping forces develop that, in severe cases, could cause grounding instability. The line of the keel blocks can be different from the line of the track. Thus, the slope of the track may be gentle or steep to suit local conditions, yet the vessels are lifted on practically an even keel.

Hauling Machinery and Chains

Hauling up the inclined track is accomplished by means of powerful machinery and chains, which pull the cradle and its superimposed load on a system of rollers or wheels. Some of the smaller railways use wire rope winches with multipart wire rope in place of chain. The following discussion focuses on the chain haul system and associated machinery.

Developed especially for railway dry docks, hauling machines consist of an electric motor that drives a speed reducer and a train of gears. The gears turn one or more toothed chain wheels, or sprocket wheels, driving the hauling chain. A schematic of a typical hauling machine is shown in Fig. 10-23. Unlike the cradle, track, and foundation, where local concentrated loads can greatly affect the design, the hauling machine and chains are sensitive only to the total live and dead loads. The load in the chain is entirely a function of the cradle load, the gradient, and the friction and does not come from the driving motor or engine. Basically, the load in the chain may be expressed as $W\sin\theta + WC_f$, where W is the total weight of the

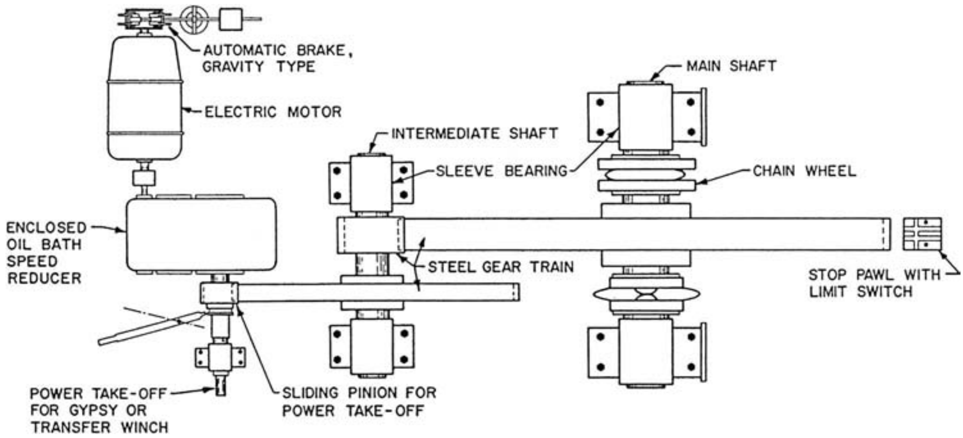


Fig. 10-23. Schematic of a railway hauling machine

maximum design vessel plus the weight of the cradle and chains, θ is the angle of inclination of the track in degrees, and C_f is the coefficient of friction.

For a roller system, the design value for the coefficient of friction ranges from 0.015 to 0.020. Lutts (1955) reports that in actual tests completed at the Boston Naval Shipyard on marine railway No. 11, the average coefficient of friction for an existing roller system was calculated to be about 0.007. For a wheel system, a coefficient of about 0.05 is used for design.

The horsepower required is a function of the pull times the speed. Whenever possible, the machine is designed to have a hauling speed of about 0.3 vertical m (1 vertical ft) per minute. For example, if a railway is built on a gradient of 1:22, then the ideal hauling speed would be 6.6 m (22 ft) per minute.

With regard to safety features, automatic brakes protect against runaway cradles, which may be the result of human error or electrical failure. Once it is in the full-up position, cradle latches secure the cradle. Also, to protect the machine as well as personnel around it, electrical interlocks are incorporated into the control system to prevent its inadvertent operation.

Chains are far more satisfactory than the best wire rope because of their enormous tensile strength, durability, economy, and ease of connection with special hauling shackles. Railways built today use Class 3 or oil-rig-quality chain. This welded, high-strength, alloy chain is about 50% stronger than cast steel chain and has a working load that is about 40% of the chain's breaking load. Hauling chain can be easily gauged for stretch or checked with calipers to determine its remaining strength after wear has occurred. Chains last up to 20 years in a saltwater environment with little or no maintenance.

From the cradle, the chain goes to the chain wheel, where it turns 180° and then extends back down the track, and the bitter end is connected to a smaller down-haul or backing chain. This chain passes through an underwater sheave secured to the

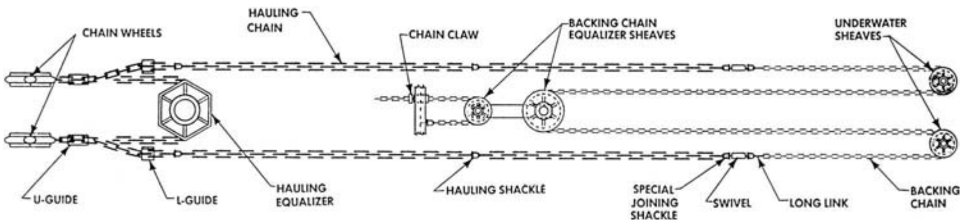


Fig. 10-24. Chain arrangement for a two-chain machine

track and back to the cradle to form an endless loop. This arrangement is illustrated in Fig. 10-24. The endless-loop system provides a means of pulling the cradle down the track if the friction force caused by silt and light debris on the track cannot be overcome by gravity.

Rollers and Wheels

For medium- and large-capacity railways, the cradle runs along the track on a system of rollers. A typical roller system is similar to a longitudinal roller bearing and demands the same precision, clearance, and perpendicular alignment. Present-day roller systems consist of cast iron, ductile iron, or cast steel rollers held in position by steel frames consisting of angles with welded-in malleable iron bushings to receive the roller pintles, as shown in Fig. 10-25.

The roller width and spacing are dictated by the maximum lineal load. The minimum roller width for a modern railway is 150 mm (6 in.), whereas the largest railways to date have 355-mm (14-in.) wide rollers. Each roller frame holds about 12 individual rollers spaced at approximately 300 to 450 mm (12 to 18 in.) on center. Frames are connected together, forming a continuous length of rollers on each way.

Some railways are built with wheels. Wheels of the fixed-axle type have approximately 2 1/2 times the friction of rollers and so require heavier chain and slightly more powerful machinery than rollers.

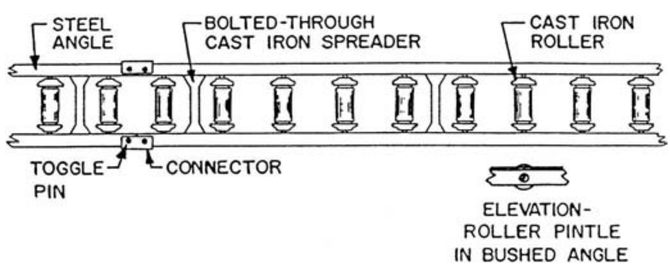


Fig. 10-25. Marine railway roller system

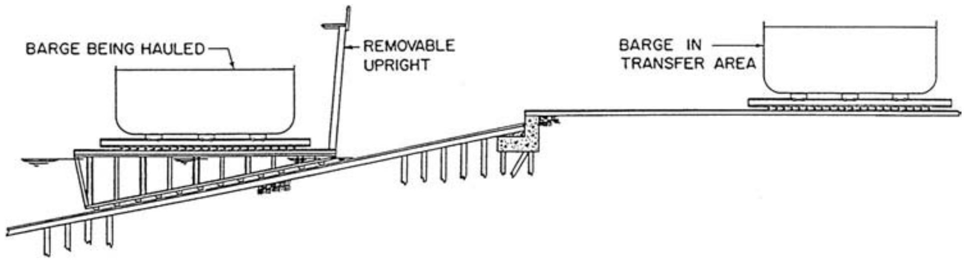


Fig. 10-26. Side-haul railway

Side-Haul Railway Dry Docks

A marine railway side-haul arrangement is most commonly used at river sites, where there are wide ranges of water levels. Periods of low water may last several months but may be interspersed with shorter periods of high water. The typical vessels navigating such rivers are shallow-draft, flat-bottomed craft of light construction. The typical side-haul railway, as shown in Fig. 10-26, is constructed on a relatively steep gradient, providing sufficient vertical movement to allow docking at all river stages. In general, the side-haul railway is expensive to build and is not well suited to oceangoing vessels. The side-haul railway should be chosen only if careful study indicates that other types are unsuitable.

A side-haul railway typically consists of several cradles, each handled by one or two parts of chains or wire ropes. The principal technical disadvantage is that the load on any one cradle section is statically indeterminate and varies according to the location of the ship's longitudinal center of gravity. Consequently, it is necessary to use oversize chains or wire ropes to allow for overload. Furthermore, the heavily loaded wires may stretch, causing differential movement of the cradle. This movement may be overcome by using chains driven by sprocket wheels running at the same speed (Crandall Dry Dock Engineers 1979, 1980).

Vessel Transfer on a Marine Railway

Adding to the versatility of marine railways is the ability to transfer vessels from the cradle to berths on shore or vice versa. The advantage gained in having the cradle deck horizontal in the full-up position is fully realized when transfer is involved. Ship transfer systems have become extremely popular because of needs created by the prefabricated, modular-type assembly line techniques used in modern ship construction. Also, the economic benefits of handling several ships for long-term repair jobs as well as the possibilities for winter storage are among the reasons why owners choose to incorporate a transfer berth or berths in the original construction or to make allowance for such installation at a future date.

The relatively low cost of a transfer system that uses a single railway dry dock as the basic lifting and launching facility makes transfer advantageous for large-scale developments. The cost varies greatly, depending on the arrangement, number of berths, and selectivity required, but, in general, one transfer berth, including a transfer cradle, costs from 25% to 30% of the cost of the railway itself.

The principles of rolling and hauling used for the inclined railway can be applied to the transfer of a vessel on a horizontal plane. The difference is that no component of gravity needs to be overcome, only the friction forces. The effort required to move a ship horizontally is only a fraction of what it takes to pull it up an incline.

The typical transfer system consists of a pair or a series of ways bearing on concrete footings or pilings. The ways support the transfer cars, which in turn support the ship. Blocking is provided between the transfer car and the ship to give headroom and clearance for working on the hull. It also serves to distribute the vessel weight as much as possible. The transfer cars are fitted with wheels or travel on a roller system.

The choice between side transfer and longitudinal transfer is largely a matter of convenience with respect to the various conditions of the site. Longitudinal transfer off the end of the cradle has proved to be economical, especially if only a single way is used. However, it does require that the entire hauling machine be depressed in a pit below the level of the transfer rails. When full selectivity of storage space is desired, then two-directional transfer is required. In this case, the cost of end transfer is about equal to the cost of side transfer. With two-directional transfer, the second direction of movement is accomplished at a lower level than the initial transfer. This arrangement permits the car for one direction to be superimposed on the car for the other direction. Figs. 10-27 and 10-28 show typical transfer schemes for a marine railway.

The ability to transfer vessels on a marine railway has enabled some yards to abandon the use of greased ways for launching ships and to adopt the safer method of controlled launching, with the added advantage of being able to construct the vessel from a horizontal work plane.

For additional information on the features, characteristics, and general guidelines for design of marine railways, see BSI (1988), Crandall Dry Dock Engineers

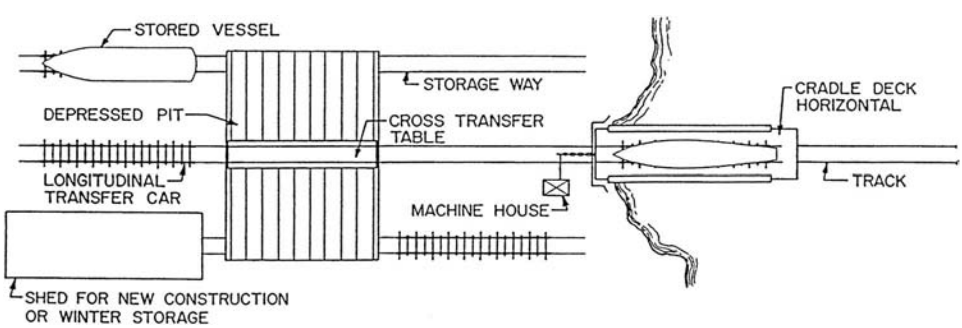


Fig. 10-27. Marine railway with longitudinal transfer

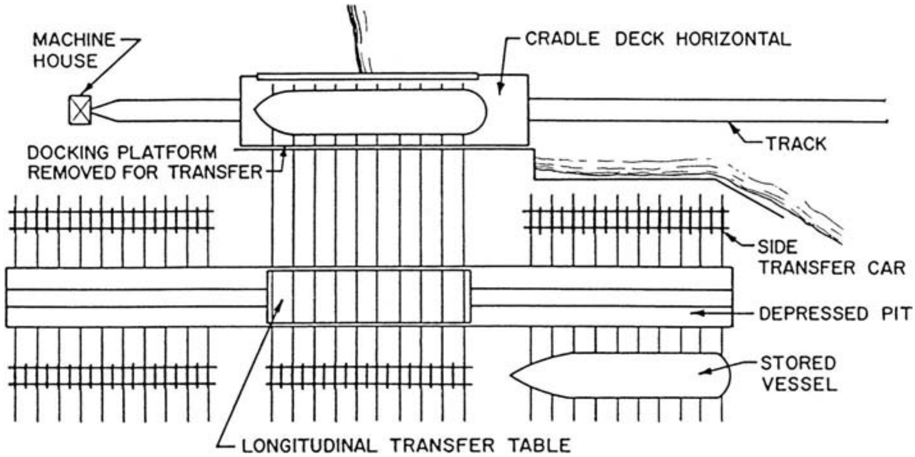


Fig. 10-28. Marine railway with side transfer

(1980, 1989), Mackie (1979), and Heger (2005). For information on the care and maintenance of marine railways, see Crandall Dry Dock Engineers (1979).

10.4 Floating Dry Docks

Floating dry docks are floating structures with sufficient dimensions, strength, displacement, and stability to lift a vessel from the water. They typically consist of two main parts: pontoons and wing walls (Fig. 10-29). The pontoon (or pontoons) is the prime supporting body, which must displace the weight of the vessel and the dry dock. It also must be able to withstand the transverse bending caused by the ship's weight along the centerline opposed by the water pressure from beneath. The wings provide stability while the pontoons are submerged, as well as space for equipment on the wing deck and inside the wing structure. Part of the wing structure is used for ballast water, needed to sink and control the depth of submergence of the dock. Continuous wing walls serve as longitudinal girders. Most dry docks are rectangular, although some are shaped more like ships at the pontoon bottom.

Floating dry docks may range in size up to 100,000 tons lift capacity, with lengths more than 300 m and widths greater than 60 m. They have been constructed of wood, iron, steel, reinforced concrete, and a combination of wood and steel. Almost all modern floating dry docks are of all-welded steel construction.

The earliest reported use of a floating dry dock occurred around 1700 in the tideless Baltic Sea. In order to effect repairs to his vessel, a ship's captain acquired the hulk of a larger vessel, the *Camel*, removed the decks and bulkheads, and cut off the stem. After floating his own vessel into the larger hull, he bulkheaded the stem and removed the water from inside the larger vessel. The *Camel*

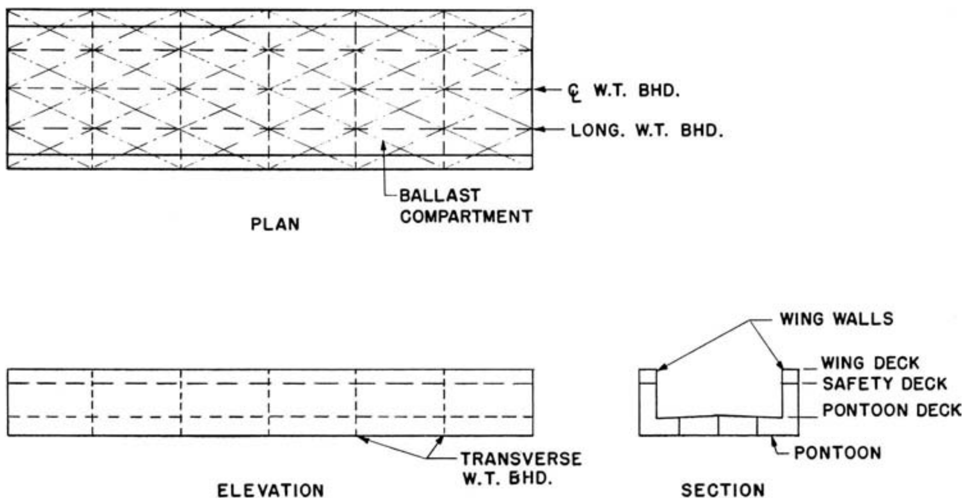


Fig. 10-29. Major features of a floating dry dock

floated with the smaller vessel inside, high and dry and with the hull accessible for repairs.

For some time after this, floating dry docks were either made from, or made in the shape of, vessel hulls. Gradually, the dry docks were made in U-shapes and rectangular or box shapes toward the middle and latter parts of the eighteenth century. At first, only wood was available for construction, but the introduction of iron and eventually steel as construction materials provided improved capabilities and increased sizes in floating dry docks.

As vessels grew larger and heavier, floating dry docks followed suit, with a variety of designs. Maintenance of the dry dock by virtue of a self-docking capability was an important feature. Naval powers, particularly England and Germany, required mobility of their dry docks to service their large fleets scattered around the globe. These two factors contributed heavily to the various types of floating dry docks developed through the years.

Today, floating dry docks continue to be important parts of many ship repair facilities. Through the use of improved protective coatings and cathodic protection systems to extend useful life before repairs or dry-docking are required, modern floating dry docks are most often built as non-self-docking, one-piece steel units.

Floating dry docks are the most flexible type of repair facility available, in that they can accommodate vessels of all sizes, up through supertankers and very large crude carriers (VLCCs), and they represent an asset that can be readily relocated as conditions require. They are able to operate with a list and/or a trim to accept vessels floating at almost any attitude. Because the dry docks float adjacent to a pier or dolphins, they can be located beyond pierhead lines and do not occupy as much valuable waterfront real estate.

Types of Floating Dry Docks

Floating dry docks may be categorized in several ways. They may be divided into those that are intended to be self-docking and those that are made as one-piece units and require another dry-docking facility for maintenance and repairs. The self-docking designs generally use removable sections that fit on the remaining portions of the dry dock. Through a cycle of self-docking operations, the entire dry dock can be serviced.

The first such structure was developed by Rennie in the mid-1700s, using sectional pontoons with continuous wing structures. An individual pontoon unit could be disconnected from the wing structure and lifted by the remaining pontoons for maintenance and repairs. This was an important feature as it helped to extend the serviceable life of a dock. This type of dry dock is still in use.

Variations of the Rennie concept include dry docks that are completely sectional with several one-piece wing and pontoon units joined together. An individual unit can be disconnected from the adjacent unit(s) and lifted by the remaining units for service. Other types use two relatively short end pieces and a long center section as an operating unit. The short end units can be dry docked on the longer center unit, and the two end units can be used to lift the center section. Several of these docks are still in existence from their World War II days.

Earlier sectional concepts used removable connections along the vertical face between the wing structure and the pontoon units, which permitted the wings to lift the pontoons and the pontoons to lift the wings for service. This design concept, as well as several others, is shown in Fig. 10-30.

All sectional dry docks exhibit inherent weakness at the connections between sections. The strongest dry dock with the most efficient use of building material is the one-piece, continuous-box dock.

Floating dry docks also may be differentiated by their shape (i.e., ship-shaped or trough-shaped). The ship-shaped floating dry dock is best exemplified by the U.S. Navy's auxiliary floating (ARDM, formerly ARD) dry docks of World War II vintage. These are essentially floating graving docks with closed bows and stern gates. The vessel to be dry-docked must fit completely within the confines of the dock. Lift is provided by the displacement of the dry dock structure and the evacuated cavity within that structure. Although not self-propelled, their shape allows them to be readily towed to advance bases. The pontoon has less depth than a trough-shaped dry dock of similar lift capacity because the working or pontoon deck is below the outside water level, requiring less operating depth and possibly less dredging at the operating site.

The trough-shaped dry dock, which must have its working deck above water level, requires a deeper pontoon and operating site for an equivalent capacity, compared to the ship-shaped type. Because the ends are open, a docked vessel may exceed the length of the dry dock and overhang the ends of the pontoon. These docks can be relocated, but tow preparations for this type of facility usually are more involved than those required for the ship-shaped dry dock.

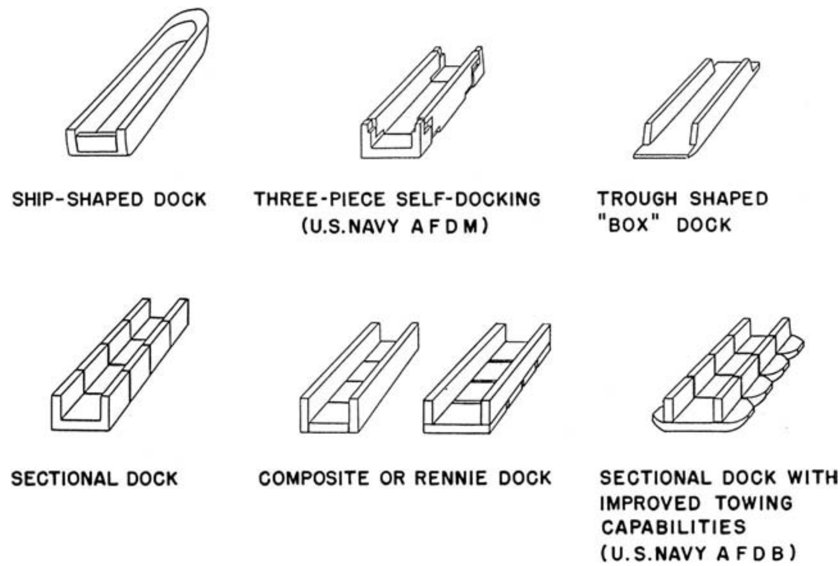


Fig. 10-30. Types of floating dry docks

Source: Adapted from Amirikian (1957)

Another way to categorize dry docks is by the materials used for construction. Originally, all floating dry docks were constructed of wood. Iron, then steel as well as reinforced concrete gradually replaced wood as the favored construction material. Composite construction, usually consisting of wooden or concrete pontoons and steel wings in a Rennie or sectional design, produced a sound dry dock capable of many years of useful service.

Wooden dry docks were limited in their dimensions and load-carrying capacity, so as ships became larger, the wooden dry dock became less practical. Wood was used extensively in World War I and World War II because of a desperate need for dry docks and a shortage of steel, but no wooden dry docks have been built since then. Difficulties in obtaining good-quality wood in the dimensions required and the fact that working with large timbers is a lost art, as well as the increased size of dry docks and the capabilities of steel construction, have led to a discontinuance of wooden dock construction. Wood is a natural construction material for saltwater because of its life expectancy in that preserving environment; as long as the wood was kept wet by occasional operation and marine borers could be repelled, it would be preserved. However, exposure of some areas of the wing walls to rainwater and the destructive capabilities of marine borers in the pontoons led to the demise of most wooden dry docks.

Composite construction, incorporating wooden pontoon units and steel wing walls, used the advantages of wood's self-preservation in seawater and the strength and survivability of the steel above it. Many of the all-wood dry docks were repaired by replacing the decayed wing walls with steel wing boxes. This method helped to greatly extend the useful life of the facility, and a few of these structures are still in operation today. No new composite docks have been built, for reasons cited previously.

Reinforced concrete was used for construction of several floating dry docks for the U.S. Navy during World War II, largely because of the lack of steel during the war period. These docks were smaller, with lift capabilities between 2,800 and 4,000 tons. Reinforced concrete docks are more common in the former USSR countries. A 100,000-ton lift capacity concrete dock was built in Italy and moved to Turkey in the late 1990s (Giuffre and Pinto n.d.).

Concrete docks have a high deadweight-to-lift ratio, requiring deeper pontoons than steel docks of the same lift capacity (Fig. 10-31).

Additional information on the history and types of floating dry docks can be found in Cornick (1958), Cunningham (1910), Crandall Dry Dock Engineers (1977), Cook (1957), and Thatcher (1978/1979).

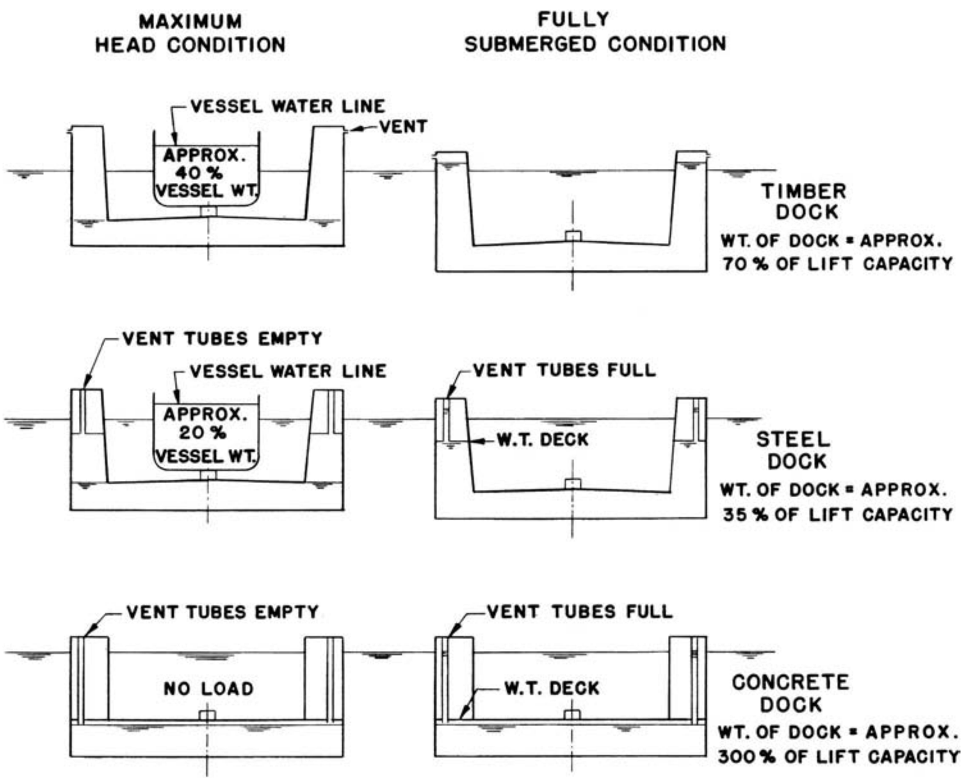


Fig. 10-31. Comparative water levels and head conditions for dry docks constructed from different materials

Source: Adapted from Crandall Dry Dock Engineers (1977)

Fig. 10-32. Modern floating dry dock at Norfolk, Virginia

Source: Photo courtesy of Heger Dry Dock

Today, the use of all-welded steel construction provides an efficient relationship among the weight, strength, and lift capacity needed for modern ship repair facilities. Shipbuilders or fabrication yards dedicated to building floating dry docks using modern shipbuilding techniques generally build these docks. Fig. 10-32 shows a modern floating dry dock. Several of the equipment items mentioned in Section 10.1, such as blocks, cranes, and traveling stages, are evident in this photograph.

Principles of Operation

A floating dry dock works on the basis of the Archimedes principle, displacing a volume of water equal in weight to its own weight. The dimensions and arrangement of the structure, and the presence of the pumping and flooding system, permit the weight of the dry dock to be varied so that the dock's pontoon deck may be submerged to dock or undock a vessel. Water is added to the ballast tanks, sinking the dock until the desired water depth is attained. The vessel is then placed in, or removed from, the dry dock. Water is then evacuated from the ballast tanks, and the dry dock rises from the water until the pontoon deck is above the water level.

The capacity of a floating dry dock may be limited by its buoyancy, its stability, and/or the strength of the structure. Generally, the buoyancy is the limiting condition because sufficient stability and strength should be designed around the buoyancy. There are, however, exceptions to this general rule, involving both stability and strength.

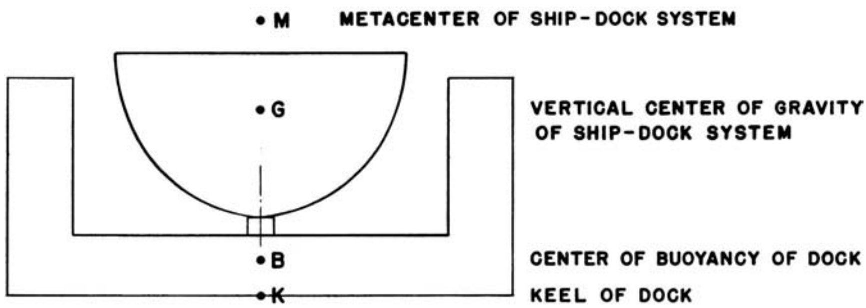


Fig. 10-33. Important distances for ship-dock stability calculations

Intact stability must be assessed for all phases during a docking operation. Stability is generally investigated for five separate phases of the docking or undocking evolution:

- Phase 1—Dock at full submergence without ship
- Phase 2—Partial lift of ship: vessel has been lifted approximately half of its draft
- Phase 3—External waterline at the top of the keel blocks
- Phase 4—External waterline just above the pontoon deck
- Phase 5—Dock at normal operating draft

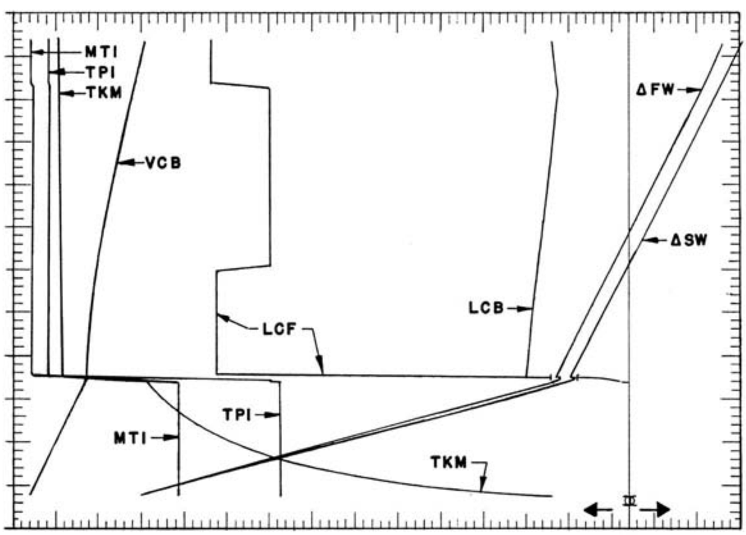
Fig. 10-33 shows the important elements for determining stability for a ship-dock system. The distance between the points *G* and *M*, designated *GM*, is a measure of a floating body's stability. *G* is the vertical center of gravity of the floating body. For a dry dock, this measure includes the weight of the dock, the weight of the ship, and the weight of the ballast water and any other equipment, fuel, stores on the dock. *M* is the metacenter which is the point in which vertical lines through the center of buoyancy pass for small angles of list. The location of *M* above the keel is a function of the waterplane area cut by the dock at any particular draft. If *GM* is positive (i.e., *M* is above *G*), the floating body is stable. If *GM* is negative (i.e., *M* is below *G*), then the floating body is unstable and may take a large list or may capsize.

In order to ensure adequate stability, a floating dock should maintain a minimum positive *GM* throughout all phases of the lift. For docks with a lift capacity of 10,000 long tons or less, the minimum *GM* should be 1.5 m (5.0 ft). For docks with a lift capacity of 50,000 long tons or greater, the minimum *GM* should be 1.0 m (3.28 ft). For docks with a lift capacity between 10,000 long tons and 50,000 long tons, the minimum *GM* should be interpolated (based on capacity) between 1.0 and 1.5 m on a straight-line scale (ABS 2009).

In the minimum stability phases (Phases 3 and 4), only the wing walls cut the waterplane and provide stabilizing force. Critical stability occurs after the vessel's keel comes out of the water but before the pontoon deck breaks the water's surface. As the dry dock rises and lifts the vessel from the water, the vertical center of gravity (v.c.g.) of the ship-dock system rises. When the vessel's keel breaks the water's surface (Phase 3), the center of gravity of the system is very high, and the positive inertia, now being provided only by the dock's wings, is at a minimum. Until the pontoon deck breaks the water surface, stability is at a minimum. This is the critical phase of intact stability, and the dimensions of the wing walls and the ballast tanks must be coordinated with the dock's design vessel(s) to ensure positive stability characteristics for all potential combinations of vessel weight and vertical center of gravity.

The size of the ballast tanks also plays an important role in determining stability. The wider ballast tanks are, the more the internal ballast water can shift to one side of the tank should the dock take on a small list. This shifting of ballast increases the list in the dry dock and reduces the overall *GM* of the system. This free surface of the ballast tanks and other tanks, such as fuel or potable water, must be accounted for in the stability calculations. Refer to Section 9.2 for a discussion of the stability of floating bodies. Information on the hydrostatic and stability characteristics of a floating dry dock are provided on the curves of form and stability curves, as shown in Figs. 10-34 and 10-35.

When undocking a vessel, the ship–dock system goes from having a large *GM* (very stable) with its pontoon deck out of the water to its minimum stability phase once the pontoon deck goes underwater. This is called the "multiplication effect" and is important because any list the dock has before submerging the pontoon deck



MTI : MOMENT TO TRIM ONE INCH
TPI : TONS PER INCH OF IMMERSION
TKM : TRANSVERSE METACENTRIC HEIGHT
VCB : VERTICAL CENTER OF BUOYANCY
LCF : LONGITUDINAL CENTER OF FLOATATION
LCB : LONGITUDINAL CENTER OF BUOYANCY
ΔFW : TOTAL DISPLACEMENT IN FRESH WATER
ΔSW : TOTAL DISPLACEMENT IN SALT WATER

Fig. 10-34. Hydrostatic curves for floating dry dock

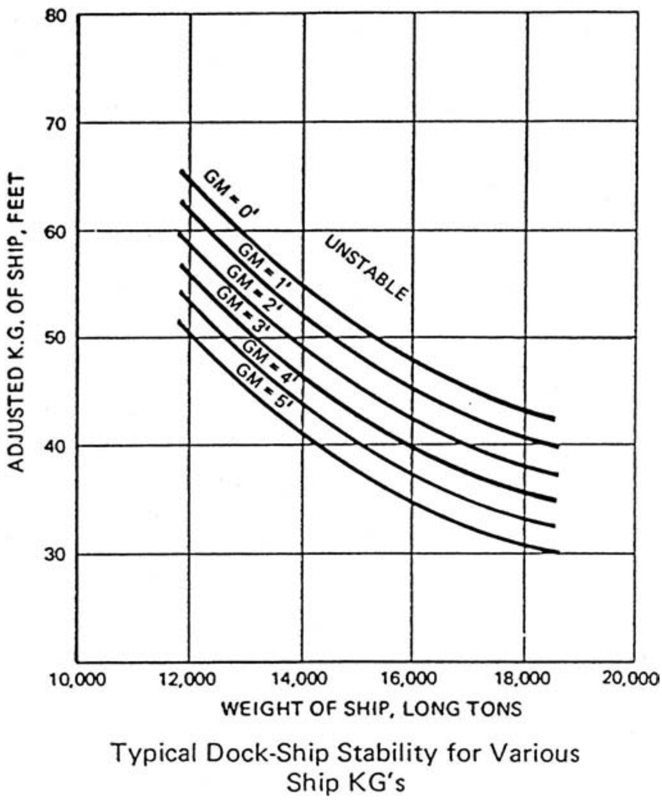


Fig. 10-35. Intact stability curves for floating dry dock

Source: Adapted from Crandall (1987b)

is multiplied by the ratio of the *GM* with the pontoon deck out of the water to the *GM* of the dock with the deck submerged. Any list present in the dock's attitude as it passes through this phase is compounded by this ratio [e.g., a 150-mm (6-in.)] list becomes a 600-mm (24-in.) list for a dock with a multiplication effect ratio of 4:1 as it submerges). This effect is of concern only as the dock is submerged, not during the lift of a vessel, because when lifting, the effect is the inverse of the ratio and list becomes smaller as the pontoon deck comes out of the water. The dock operator may slow the flooding rate and/or operate the dock on a longitudinal trim to reduce the transition rate and minimize the effect of the multiplication effect during the operation. The dimensions of the dry dock determine the ratio and the effect of this phenomenon.

Requirements for ship repair yard operators desiring to dry dock U.S. Navy vessels in floating dry docks demand stability analysis for the dry dock in a damaged condition. The impact of this requirement on the design of the dry dock is that the structure needs more compartmentation and larger wing walls than would be required for intact stability considerations alone. Buoyancy, structural, or intact stability limitations may be secondary to the restrictions on safe docking capacity dictated by these damage-stability requirements (U.S. Navy, Naval Sea Systems Command 2009, Wasalaski 1982). The U.S. Navy damage-stability requirements are designated in MIL-STD-1625D, *Safety Certification Program for Drydocking Facilities and Shipbuilding Ways for U.S. Navy Ships* (DOD 2009).

The second situation in which buoyancy limitations are not the controlling factor in design involves those dry docks designed with the capability of transferring vessels between the dock and the shore. Often, when the purpose is to use the dock to transfer vessels onto shore, it is necessary to lift vessels to a higher freeboard than would be required for a conventional dry-docking in order to get the pontoon deck to yard grade. To accomplish this goal, additional pontoon depth is needed. For economy of design in the strength and weight of the structure, the structural capacity of this extra-deep pontoon may be less than the actual buoyant capability. Thus, the structure is the limiting consideration in the capacity of the dry dock.

Compartmentation of the pontoon provides for more precise control of the dry dock, in addition to enhancing the stability. Differential deballasting of the dry dock's ballast compartments permits a more equal and opposite reaction to the loading imposed by the vessel, reducing stresses in the dry dock and vessel structure during a dry-docking. Also the dock's attitude can be adjusted by differential deballasting to accommodate a vessel's list and/or trim.

The load distribution of a vessel can be estimated as shown in Fig. 10-36. The analysis treats the vessel as a rigid beam resulting in a trapezoidal load distribution and requires the vessel's displacement, the location of the longitudinal center of gravity, and the location and length of the keel bearing on the blocks. More detailed information, such as that provided by a typical naval architect's weight distribution,

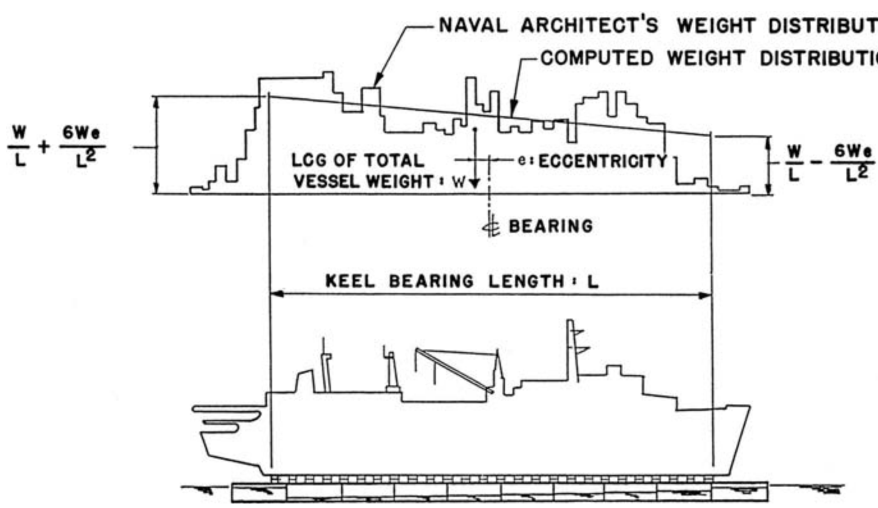


Fig. 10-36. Determination of vessel load distribution

usually is not required for standard dockings because of the elasticity of the blocking system between the vessel and the dry dock, as well as the strength and rigidity of vessels and dry docks. More involved docking situations, such as those created by damaged ships or in cases where the ship is to be cut while in dry dock, require a more detailed analysis. More complex ship hull configurations, resulting in large overhangs and unsupported lengths of the vessel in dry dock, have created concerns about the validity of this simple analysis, see Taravella (2005) and additional references cited near the end of Section 10.1. Analysis of stresses in floating dry docks caused by a docked ship are presented by Potvin et al. (1969) and Vaughan (1966).

Ballasting and dewatering of the dry dock is accomplished through an arrangement of pumps and valves. Pumping or flooding operations should be completed in 1 to 3 hours. A floating dry dock may be described as pump-controlled or valve-controlled.

A pump-controlled dock uses a large number of small pumps and valves, as each ballast compartment typically has its own dewatering pump and flood valve. The rate of dewatering for each ballast tank is controlled by starting and stopping the pump. A cross connection is provided to join adjacent ballast compartments together so that if one pump fails, the adjacent pump can be used to dewater two ballast tanks. The pump-controlled arrangement can be beneficial in areas where sediment accumulation in the ballast tanks may be significant. The ballast tanks can be flushed by flooding and dewatering at the same time to stir up the accumulated sediment and discharge it. This type of arrangement may not be practical for pontoons with three or more ballast tanks across their width because of difficulties in reaching the inner tanks and the redundancy needed in the piping system.

A valve-controlled dry dock uses a few large pumps to dewater the ballast compartments through a manifold arrangement of piping and valves. The rate of dewatering is controlled by adjusting the valves to each tank while the pump runs continuously. A few large flood inlets also are used to supply ballast water to the many tanks via the piping arrangement. This type of arrangement is well suited to docks with three or more ballast tanks across their width. Figs. 10-37 and 10-38 illustrate typical pump and valve arrangements for the two concepts.

Design Considerations

A dry dock generally is intended to service a specific group of vessels, which determines the design criteria. Any vessels whose requirements do not exceed those of the design vessel may be serviced in the dry dock.

The area of the pontoon deck must provide sufficient working space under and around the vessel for access by shipyard workers and their equipment. The wing deck and safety deck areas must accommodate the equipment to be mounted on or in these areas. The pontoon must be deep enough to provide the required lift capacity for the design vessel, and the wing walls must be sufficiently high to achieve the proper draft, so that vessels can enter and leave the dock over the blocking.

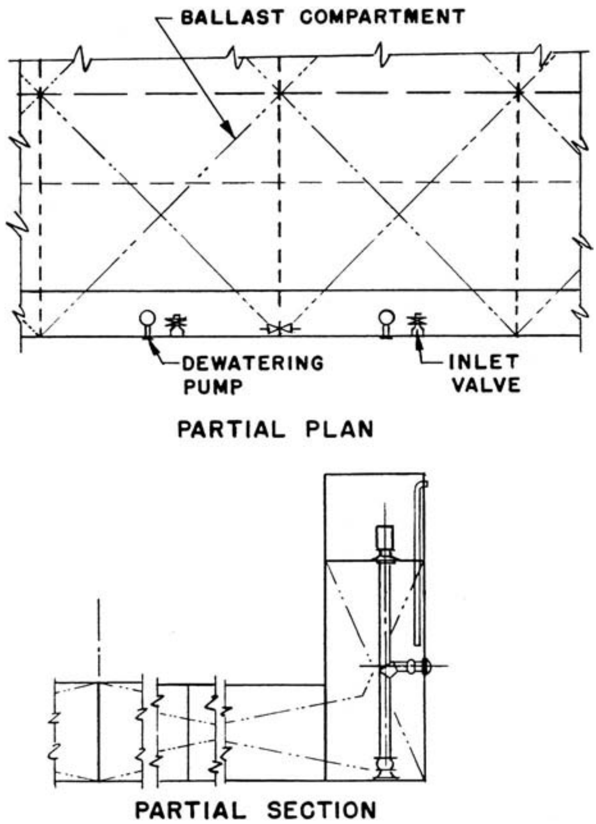


Fig. 10-37. Pump and valve arrangement for pump-controlled dock

The wing walls must provide the required moment of inertia and buoyancy for conditions of minimum stability and full submergence. The compartmentation of the pontoon must provide adequate stability and buoyancy control for the safe operation of the dock. The transverse strength of the pontoon must adequately support the concentrated vessel loading along the keel line and distribute the load across the full width of the dock to the buoyancy below. Other situations for the distribution of vessel load on the pontoon deck also must be considered, including the loading imposed by side blocks and the effect of buoyant loading with no vessel loading from above (Fig. 10-39).

Longitudinally, shear forces and bending moments must be investigated to ensure that resulting stresses and deflections are within acceptable limits. For a one-piece dock, this loading can be carried by the wing and pontoon structure. For Rennie-type docks, all longitudinal stresses must be carried by the continuous wing walls. Sectional docks cannot carry longitudinal stresses unless the sections are pinned or otherwise connected. In this case, the capacity of the connections is likely to be limited, and careful ballasting is required.

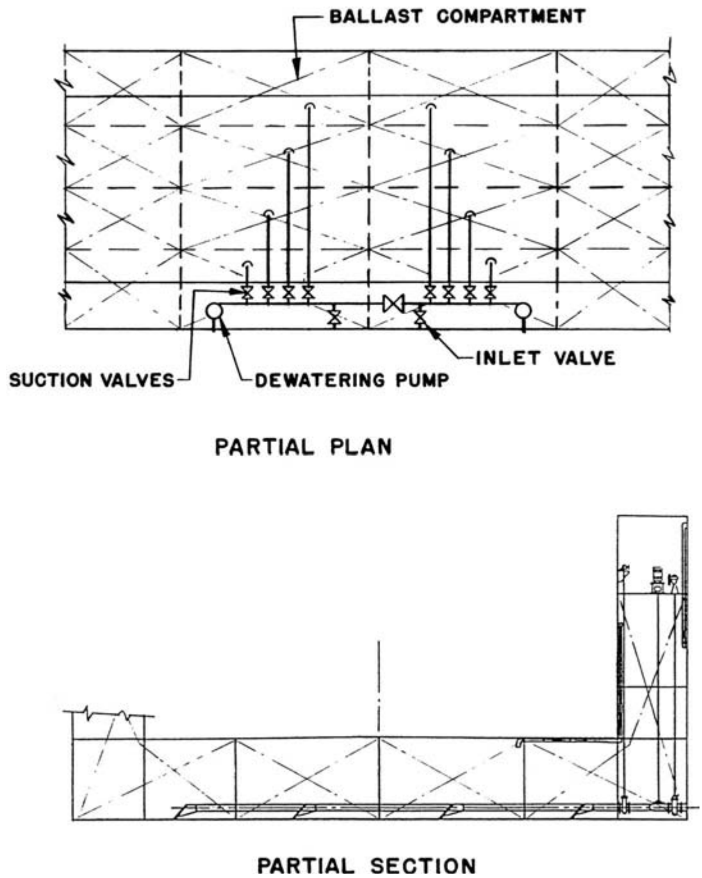


Fig. 10-38. Pump and valve arrangement for valve-controlled dock

The shell plating and the internal watertight bulkheads must be designed to withstand the head pressures experienced during the operation of the dock. A determination of the maximum head pressure encountered during a docking or undocking operation reveals the highest shell plate loading. The maximum head pressure between ballast tanks is a function of the intended submergence of the dock, the dimensions of the wing walls, and the draft and beam of the vessel to be dry docked.

Local loading conditions also must be investigated, including keel and bilge block loads and other loadings on the pontoon deck, such as vehicular traffic for forklifts or trailer trucks. Foundations for equipment to be mounted on the wing deck and safety deck, such as winches, capstans, ventilators, cleats, and so on, must be designed into the structure.

The mooring arrangement, and means of access by both vehicular and pedestrian traffic, must be considered. The dry dock may be anchored to chains in the middle of a harbor, with access via launches or an access ramp. Often, to make the

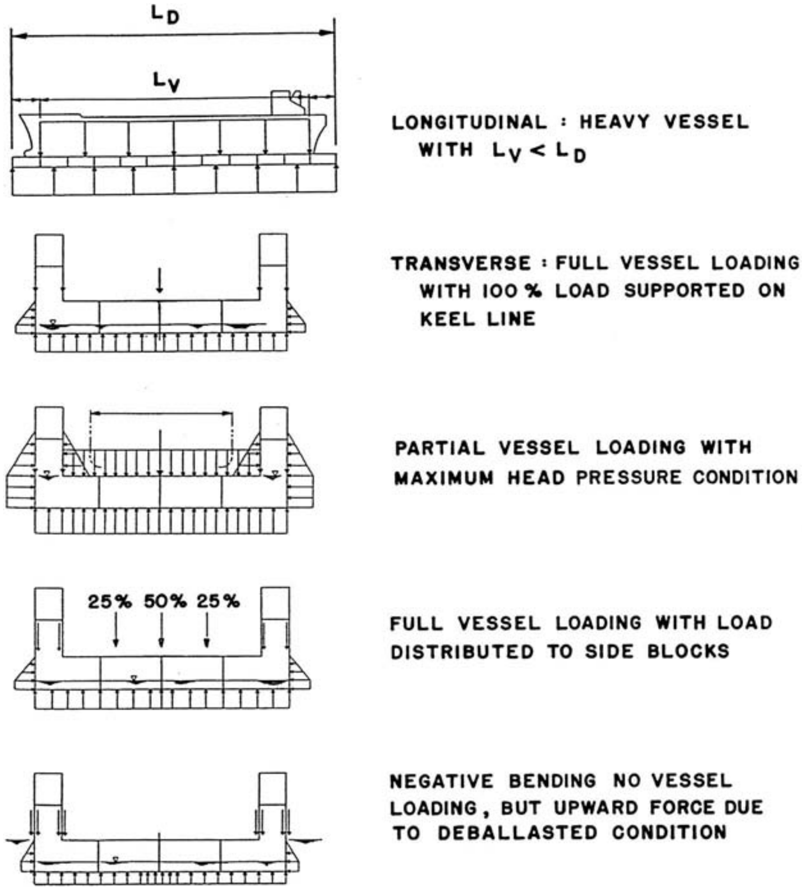


Fig. 10-39. Basic design criteria for floating dry dock

dry dock more accessible, the dock is attached to a pier or dolphins close to shore by a fixed mooring arrangement.

Design guidance can be found in the rules of several classification societies, such as the American Bureau of Shipping (ABS 2009), and Lloyd's Register of Shipping (1987), which have prepared rules for the classification of dry docks by their organizations. Additional information on the design of floating dry docks is available in Heger (2005), Crandall (1987b), Crandall Dry Dock Engineers (1977), Amirikian (1957), and Anderson et al. (1976).

Features of Floating Dry Docks

Although they are not specifically required for operation, several features considered in dry dock design offer practical advantages. Its accessibility can be a major advantage for this type of facility, as compared to a basin dry dock. The pontoon deck is at or near yard grade and, with adequate access via pedestrian and vehicle access ramps and crane service, can serve as an extension of the yard itself. The rapid and efficient movement of personnel and materials onto and off the dock can make ship repair efforts more efficient.

Remote pump and valve controls and ballast indication systems greatly increase the operator's knowledge of the status of the operation and reduce the number of people needed to control the pumps and valves. A central control station, often a control house located atop one wing wall, contains the water level indicators, valve operators, and pump controls needed to monitor and operate the dry dock. Draft indicators can be displayed and their data can be interpreted to indicate the deflection, list, and trim condition of the dock.

Alarm systems allow monitoring for fires in the safety-deck spaces, and communication systems permit communication between the control house and all areas of the wing and safety decks.

Transfer Systems

Floating dry docks can be equipped to accommodate the horizontal transfer of vessels to and from landside facilities. This accommodation may be done in either the longitudinal or the transverse direction. Typically, the dry dock operates in a deepwater or sinking berth and is translated to the shallower transfer area. The transfer area may include one or more transfer positions and often includes some means of supporting the dry dock at a fixed elevation.

Support may be provided for the entire dry dock, such as an underwater grid, or just for the end of the dock that mates to the land side. The end support can take the form of a shelf along the waterfront upon which the end or the side of the dry dock rests; it is intended to align the dock at the proper elevation with the landside transfer ways. The support need only be capable of supporting a seating load and is not required to carry the full weight of the dry dock. The differential ballasting capabilities of the dry dock are used to counteract the weight of the dry dock and the moving load. Some floating dry docks rely solely on ballast control to maintain proper elevation but use long transition beams from the shore to the dry dock to accommodate errors in the elevation of the dry dock and the shore.

The transfer capacity generally is less than the lift capacity because the pontoon deck must be lifted to yard grade, thus reducing the amount of buoyant lift applicable to vessel loading. End transfer is readily accomplished because of the open ends of the floating dry dock; a side transfer is considerably more cumbersome than end transfer, as the operation involves removal of the wing boxes on the side where the transfer is made. Fig. 10-40 schematically shows both types of operations, and Fig. 10-41 shows an end transfer operation in progress.

Several methods and mechanisms are available for the movement of vessels from shore facilities to or from a floating dry dock, including skidding, roller systems,

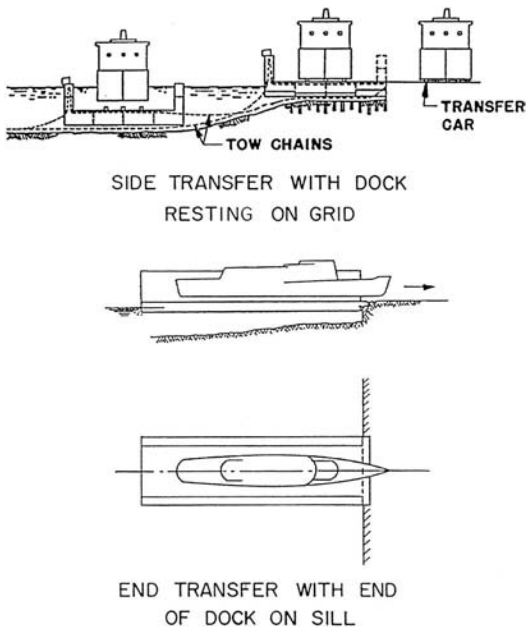


Fig. 10-40. Schematic of end and side transfers from a floating dry dock

Source: Adapted from Crandall (1975)

Fig. 10-41. Land level transfer operation onto a floating dry dock launch platform

Source: Photo courtesy of General Dynamics/Bath Iron Works Corporation wheeled systems, self-propelled modular transporters (SPMTs), air or water bearings, and walking beams.

The horizontal skidding of vessels resting on wooden blocks and cradles along greased ways is an ancient concept. Methods in use today are similar to it, with the exception of the equipment used to propel the vessel. Hydraulic jacks, used in tandem with wedged grippers that clamp onto the flanges of the transfer way, are capable of continuous motion. Skidding requires large jacking forces to overcome the friction inherent in the system.

Roller systems may use hardware similar to that of marine railways. Recirculating rollers and spherical ball units also may be used as the transfer mechanism. Roller frame systems are capable of considerable load but have the disadvantage of requiring extra length in the roller frames in consideration of the differential movement between the frame and the vessel. Recirculating rollers eliminate this problem but tend to concentrate loads to reduce the number of units required for a transfer. Spherical ball units are capable of movement in any direction but have not been widely used since it is hard to control the direction of movement.

Air and water bearings are fairly recent developments in transfer systems. Several pallet-type units are placed beneath a cradle in a pattern that counteracts the weight of the ship to be moved, and the flow of air or water creates a pressure between the pallet and the surface of the transfer area, floating the cradle with the vessel. Movement is relatively easy to achieve because of the low friction involved, although this movement could be detrimental in high winds or on surfaces with small slopes. The need for a smooth, continuous surface is a major drawback of such systems.

Walking beams have been used as a transfer mechanism for subassemblies but only rarely used for complete ships. These units are self-contained and capable of movement in any direction; they traverse slight grades and smoothly graded surfaces that have adequate bearing capacity. Their disadvantage is that they are rather complex.

The most commonly used transfer method is the wheeled system, consisting of continuous cradles or of individual cars. Continuous cradles usually are equipped with two or more rows of flanged wheels that travel on flat or crane rails. Movement is provided by an external source, such as winches with wire rope or a tractor. Individual cars usually have four wheels each and require a double track. Often, these cars are equipped with vertical jacks and are electrically self-propelled, offering additional capabilities for vessel construction. Individual *strongbacks* or cradle beams usually are used with these cars to provide flexibility. Although the continuous cradles remain with a vessel during its stay onshore, individual cars can be removed from under the strongbacks and used elsewhere. They also must be removed before submergence of the dry dock so that they do not receive water damage. Wood and/or rubber blocking may be used to support the vessel on the cradle or strongbacks. One drawback to the use of steel-wheeled cars is their requirement to travel on rails. The rails must be installed on a high-capacity foundation that does not settle over time. The alignment of the rails is critical to proper operation. As the vessel is moved

Fig. 10-42. SPMT transfer arrangement

Source: Photo courtesy of John Vitzthum

onto the floating dock, alignment of the rails on the dock must be kept within close tolerance with the rails on shore. The use of SPMTs in lieu of steel-wheeled cars eliminates much of these limitations. The SPMTs have rubber tires that can be driven on concrete, pavement, or even compacted gravel surfaces (Fig. 10-42). The many large tires distribute the load over a larger area of the ground, reducing foundation requirements. The cars can be steered to change direction. The load on each tire is hydraulically balanced such that the cars can travel over uneven surfaces (+/−300 mm) while maintaining equal load distribution. The major disadvantage of the SPMT transfer cars is their high initial cost and limited capacities. Crandall Dry Dock Engineers (1980), Heger (2005), Salzer (1986b), and Chambers (1976) provide further information on transfer systems.

10.5 Vertical Lifts

A vertical lift dry dock or *ship lift* is simply a ship elevator, consisting of a horizontal platform fabricated from transverse and longitudinal steel beams, supported by lifting machinery arranged in pairs along both sides. The platform, which supports the ship, is supported from the ends of the main transverse beams, through wire rope cables or chains connected to the lifting machinery. On most of the ship lifts still in operation today, and those currently being built, the lifting machinery consists of a series of electromechanical synchronous hoists mounted on pile-supported pier structures on each side of an articulated platform. Historically, there were ship lifts built using hydraulic cylinders and mechanical screws, such as Edwin Clark's hydraulic lift pontoon dock in 1857 at Victoria Docks, London, and in 1893, John Blackwell's 210-ft-long screw dock in Bridgetown, Barbados. There were also several ship lifts operated by hydraulic chain jacks that lifted the platform in incremental steps.

In 1957, Raymond Pearlson's invention of the Syncrolift ship lift system led to the transformation of modern, midsize shipyard design. The patented Syncrolift ship lift remains the most popular vertical lift system in the world, with lifting capacities from 30 tons up to almost 30,000 tons. Of the approximately 300 ship lift systems installed, more than 230 of them are Syncrolift systems. A comprehensive history of the Syncrolift system can be found on the Pearlson Shiplift Corporation website (Appendix 3). The largest vertical lifts still operating are Syncrolift ship lifts. One of the largest installations, in terms of overall platform size, is at Malaysia Marine and Heavy Engineering in Johor, Malaysia. The ship lift platform measures 618 ft by 111 ft, with a vertical travel of 54 ft and a maximum lifting capacity of 24,100 tons. This vertical lift was originally installed in Long Beach, California, at the former Todd Shipyard (Fig. 10-43) but was relocated to Malaysia when the yard closed. The Syncrolift ship lift with the largest lifting capacity is located in Faslane, Scotland, and is used for servicing nuclear submarines. It has a maximum lifting capacity of 25,600 tons and is one of only two ship lifts in the world that are nuclear certified.

The number and capacity of the individual lifting hoists determines the overall lifting capacity of the ship lift system. The longitudinal spacing of the units determines the maximum capacity per unit length of the ship lift that must be sufficient to lift the maximum docking load of the vessel. The vertical-lift dry dock is designed to vertically lift or lower a total load equal to the dead load of the platform, the transfer cradle, and docking blocks, plus the superimposed live load of the vessel. Schematic plans of typical ship lifts and the major components are shown in Figs. 10-44, 10-45, and 10-46.

Features of the Vertical Lift System

The horizontal, land-level attitude of the platform ideally suits vertical lifts for transfer of vessels to and from the shore. Most vertical lifts are designed with extensive onshore transfer systems that use conventional towing vehicles to move rail-supported wheeled cradles with docking blocks secured to the cradle beams (Fig. 10-47). As many as 40 onshore berths have been served by a single vertical lift. If there is sufficient land area available, a two-level transfer system with a recessed side transferring carriage can efficiently move a full-capacity vessel in less than 1 hour from the ship lift platform to the furthest repair berth in the shipyard.

Fig. 10-43. Original largest Synchrolift installation

Source: Photo by John Graham, courtesy of Todd Pacific Shipyard Division

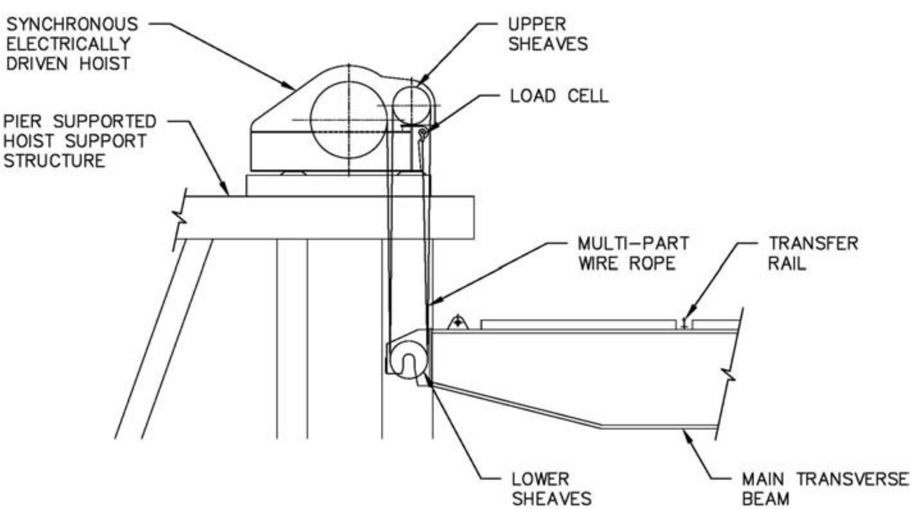


Fig. 10-44. Components of a mechanical-type lift

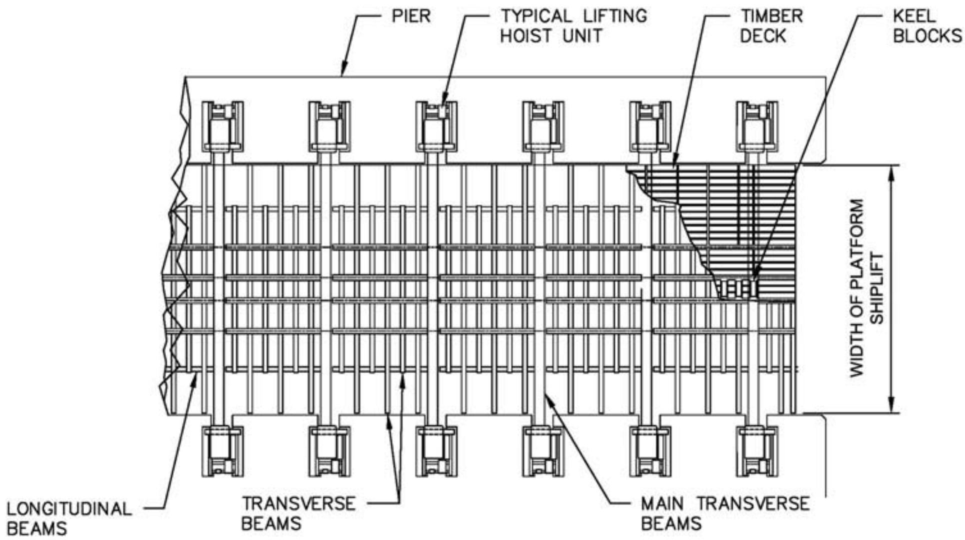


Fig. 10-45. Plan of ship lift platform

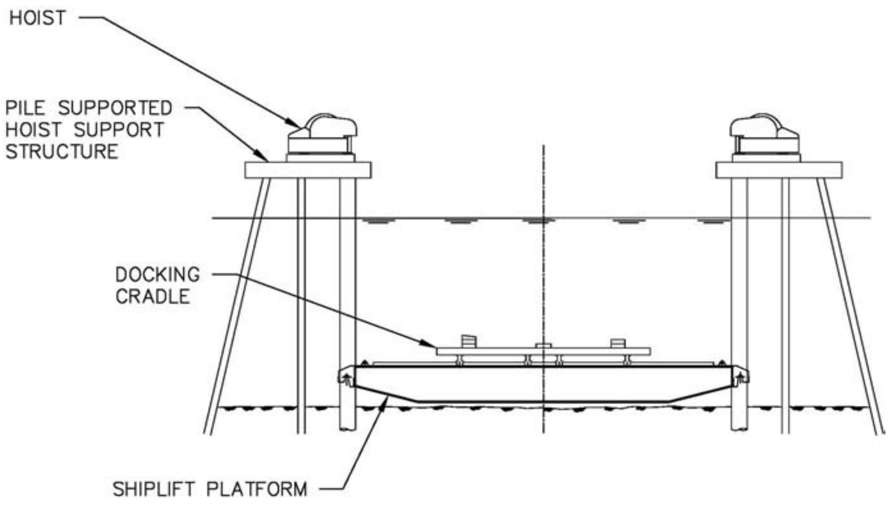


Fig. 10-46. Cross section of a ship lift

When required, docking blocks can be secured directly to the ship lift platform beams.

Material and supplies can flow naturally to a vessel on a ship lift because of the unobstructed area around the platform. The absence of vertical walls, which are present in floating dry docks and basin dry docks, permits easy access to a vessel docked on the platform to facilitate cost-effective repairs.

The speed of operation for a vertical lift is greater than that of most other types of dry docks; the average time for dry-docking a medium-size vessel in this manner is

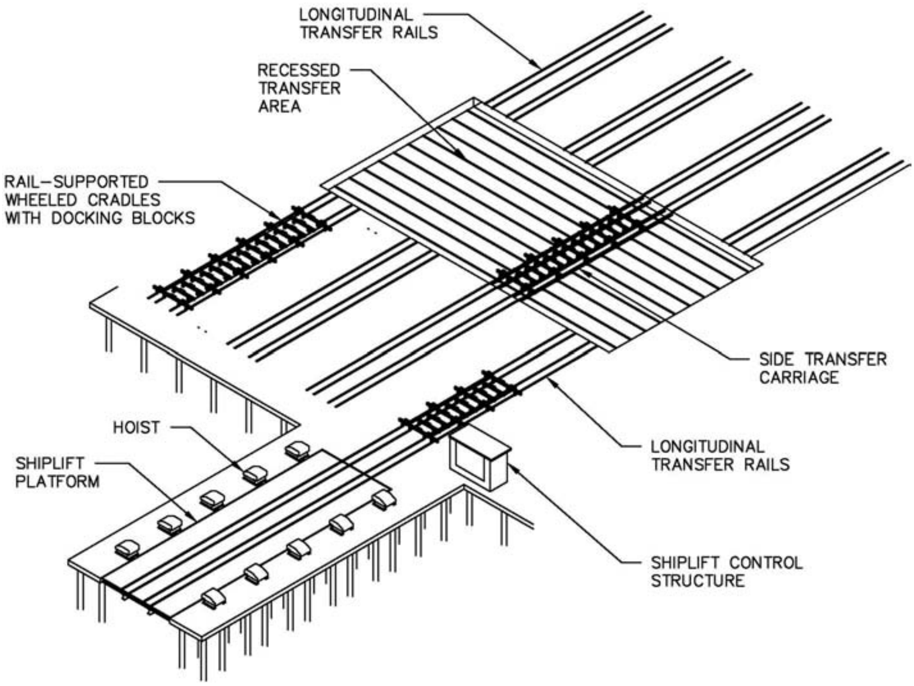


Fig. 10-47. Typical ship lift transfer berth arrangement

20 to 40 min after the vessel is secured over the submerged cradle. Furthermore, the total number of operating personnel required for a typical docking cycle is fewer than for other types of dry docks of equal capacity. The ship lift controls are usually located in a nearby structure with good visibility of the entire docking operation that can also serve as the dockmaster's office. If the ship lift uses an articulated design, the loading at each lifting unit can be easily determined with a load cell at each hoist, and the information can be displayed continuously. If required, a vertical lift is particularly well suited for lengthening and can be extended by the addition of platform sections along with an appropriate number of hoists supported on extensions of the existing piers.

Typically, vertical lift systems use electrically driven synchronous hoists with multipart wire rope cables attached to structural beams through a system of running sheaves. Most of the vertical lifts use galvanized wire rope, which should be inspected regularly because it is subject to fatigue from axial and bending stresses. Immersion in saltwater can lead to corrosion if the wire rope is not adequately lubricated to minimize wear and to protect the rope from corrosion. When it occurs between the strands, corrosion can be difficult to detect, although current nondestructive testing technologies have greatly improved this process. It is also recommended that a schedule be established for wire rope testing and/or replacement. Some systems use galvanized rope with a plastic encapsulated core. The plastic, which has a high compressive strength, forms a water-resistant coating and improves the fatigue life of the wire rope.

The motion of individual hoists must be synchronized as closely as possible to ensure that all platform lift points cover the same distance in the same period of time. This requirement is critical to ensure that the top of the docking blocks remain as level as possible to prevent overloading the hull. Synchronization is achieved through the use of three-phase alternating current (AC) synchronous induction motors, which operate at a constant speed regardless of load.

Design and Construction of Vertical Lifts

The waterfront space requirements for a vertical lift are similar to those of a floating dry dock. The actual space required is the area of the ship lift platform plus some additional space on each side for the lifting mechanism supporting the platform. The lift may be installed perpendicular or parallel to the shoreline, either fully inside the bulkhead line of the shipyard or extending out from the bulkhead. A vertical lift does not require extended space out in the water, as the incline track structure of a marine railway does. When there is a long sloping site or where the shipyard is at a high elevation, a pile-supported trestle to support transfer rails can be constructed from the shoreline to deep water to minimize dredging. This arrangement is also used to reduce the depth of the expensive deep sheet pile bulkheads at the head of the ship lift.

The lifting force required is determined by the distribution of the vessel load in docking condition. Because of load concentrations along the length of a vessel, certain hoists carry more load than others. Thus, for a vertical lift system, the lifting capacity required is not determined by the total vessel displacement, but by the maximum load concentration applied along the length of the platform. The maximum lifting capacity could then be as much as twice the ship's total displacement. This is particularly true of ship lifts designed for ships with very high load concentrations aft. In such circumstances, a dual-capacity ship lift is often used with closer hoist spacing on the shore end of the ship lift platform. Most ship lifts today are fitted with load cells at the lifting points that display the hoist load and automatically preclude the possibility of hoist overloads.

For those systems designed for transfer, the ship's movement on and off the platform may be transverse or longitudinal. The weight of the platform for direct side (transverse) transfer systems is slightly higher because an additional structure is required to support the full load of the vessel passing over one side of the platform.

There are two basic types of ship lift platform designs: continuous (often called "rigid") and articulated. With a continuous platform design, the longitudinal beams are continuous, spanning across the transverse beams so that local platform loads are spread beyond the adjacent supports (an indeterminate condition). With an articulated platform, the longitudinal beams are effectively "pinned" on at least one side of each main transverse beam, so there is no moment transfer longitudinally (a determinate condition). The longitudinal beams are framed to act as statically flexible elements between the main transverse beams. Either type of platform can be designed to be decoupled and operate as two or more separate sections for independent dockings of small ships. Furthermore, one section can be vertically offset from another for clearance of a hull projection such as a propeller or a sonar dome.

The wire-rope hoisting cables should be sized for the maximum load they will see during service plus a design factor. The design factor, which is the ratio of the breaking load of the cable divided by the working load, is specified by Lloyd's Register of Shipping (2013) to be a minimum of 3.0.

The hoisting units for wire-rope systems typically consist of small AC synchronous motors, reduction gearing, a wire-rope drum, and two independent brakes. The brakes are designed to stop the unit automatically in the event of a power failure. The sizes of the motor and the reduction gearing are generally selected to allow for a vertical lifting speed of 6 to 12 in./min.

The lifting units usually are mounted on pile-supported concrete piers along each side of the ship lift platform. In addition to supporting the lifting units, the piers can be designed for rubber-tired cranes or provided with rails for traveling cranes. The elevation at which the electric hoists are mounted must be carefully selected. They should be well above extreme high-water levels (i.e., storm tides) to avoid damage from flooding and allow the platform to be raised to a "maintenance" position clear of the normal high-water level.

Further information and guidelines for the design of structural and mechanical elements of a vertical lift may be found in Lloyd's Register's *Code for Lifting Appliances in a Marine Environment* (2013).

10.6 Mobile Straddle Lifts and Boatyard Equipment

The introduction of straddle cranes, or "mobile boat hoists," into marine service in the late 1950s revolutionized the small-craft hauling and storage industry. Boatyards were enabled to readily haul, store, and load yachts and small craft onto trailers with complete flexibility. Today, mobile straddle lifts have been constructed with lift capacities up to 1,200 metric tons, allowing them to lift commercial vessels and small ships. Generally, however, even relatively large yachts (to 20-m length or larger) can be lifted on hoists of 35 to 50 metric tons or smaller capacity. Larger capacity lifts allow the docking of commercial and service craft, including tugs and barges, and may be an attractive option to small marine railways. For larger vessels, of 100 displacement tonnage (DT) or more, special attention should be given to the vessel load distribution and the placement of slings (Tobiasson 1989).

Straddle lifts require two parallel runways, usually on pile-supported finger piers, of sufficient length and water depth between them. Fig. 10-48 illustrates the general configuration and principal dimensions of a marine straddle lift installation. Typical clear widths between piers range from 17 to 21 ft for lifts up to 75-metric ton capacity, up to 38 ft for a 500-metric ton machine, with corresponding minimum pier lengths

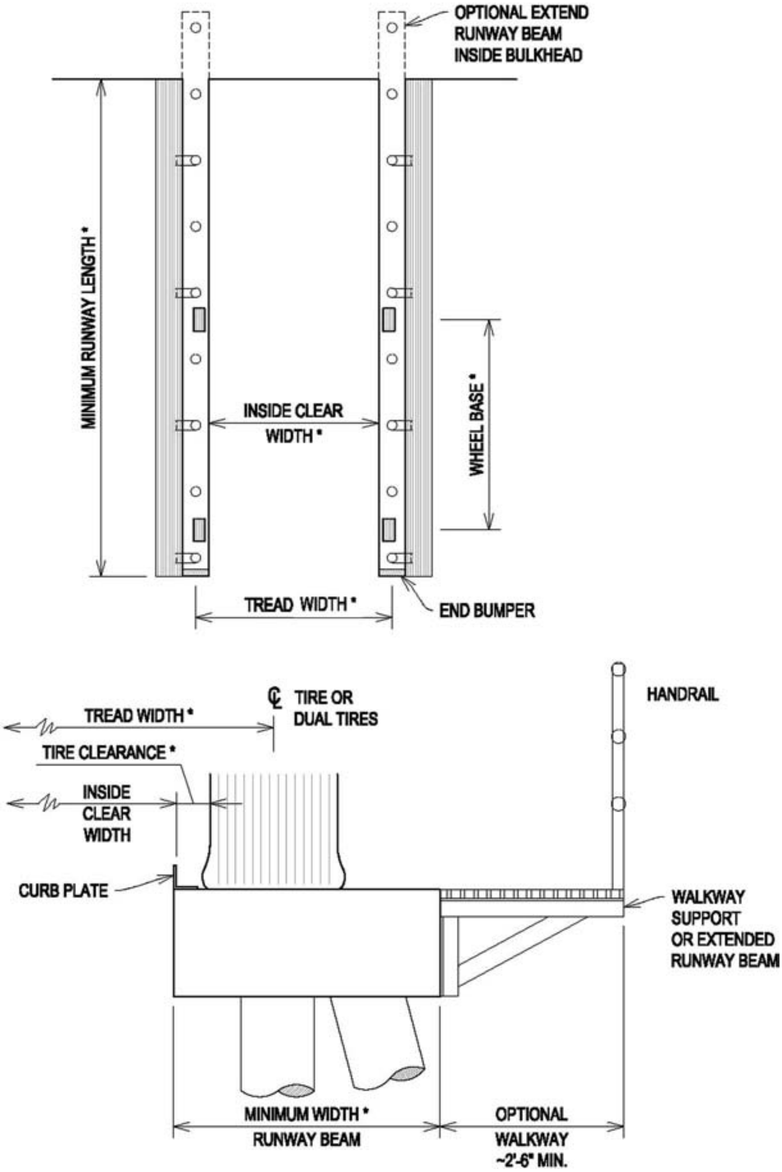


Fig. 10-48. General dimensions of typical straddle lift runway piers

from 45 to 130 ft, respectively. The vessel is supported fore and aft by slings, usually of reinforced nylon fabric, and hoisted by wire ropes reeved in several parts and equalized for load balance. The fore and aft load distribution is typically in the range of 40/60 to 33/67. Most contemporary machines are equipped with one top cross beam that can be retracted (or is omitted entirely) to allow hauling of vessels with masts or high superstructures. Runway girders may be of heavy timbers drifted together for the smallest sizes, combinations of timber and steel plate or shapes, or reinforced or prestressed concrete. They should be designed for stiffness to minimize deflections and to ensure favorable load distribution to the piles. Runway beams may extended shoreward of bulkheads or retaining structures to avoid concentrated wheel loads immediately behind the wall, as shown in Fig. 10-48. Fig. 10-49 shows a representative 35-metric ton capacity boatyard straddle lift pier installation constructed of precast concrete runway beams on braced greenheart timber pile bents. Fig. 10-50 shows a 500-metric ton straddle lift installation constructed of monolithic cast-in-place concrete runway beams on concrete-filled steel pipe piles. This installation was originally designed for a 300-metric ton machine and was able to be readily upgraded to 500 metric tons because the larger machine has two dual wheels per leg on bogies, which allow better distribution of the concentrated wheel loads. Machines up to about 100-metric ton capacity generally have one wheel per leg, machines up to about 400-metric ton capacity have dual wheels, and machines of more than about 400 metric tons have bogies or trucks with four wheels per leg. Wheel loads for a given piece of equipment must be obtained from the manufacturer. As a rough rule of thumb, single-wheel or single-leg, dual-wheel loads are usually on the order of one-half of the machine's rated capacity, to which it is

Fig. 10-49. A 35-metric ton capacity boatyard straddle lift installation constructed of precast concrete runway beams on a braced greenheart timber pile foundation

Fig. 10-50. A 500-metric ton capacity Marine Travelift straddle lift installation constructed of cast-in-place concrete runway beams on concrete-filled, rock-anchored steel pipe piles. Note the dual wheel bogies, with four tires per leg

Source: Photo courtesy of GZA GeoEnvironmental, Inc.

recommended that a 15% impact factor should be applied in the runway beam and pile caps design, consistent with NAVFAC requirements for straddle carriers (DOD 2005). Maximum tire inflation pressures are generally on the order of 110 to 125 pounds per square inch (psi) but may be as high as 150 psi for some machines. A lateral load equal to a minimum of 10% of the wheel load should be applied simultaneously with the vertical loads to the overall structure stability. Each finger pier should be capable of resisting a minimum longitudinal thrust caused by braking of 10% of the combined live load of the machine plus the vessel. Equipment weights and wheel loads must of course be verified by the manufacturer (see Appendix 3 for manufacturer websites).

Required runway widths must be obtained from the equipment manufacturer. Usually, a minimum of 3- to 4.5-in. inside clearance from the tire to the curb plate is required, and adequate allowance must also be made for minor steering corrections and possible misalignment of the hoist frame. An additional 2.5-ft minimum up to about a 5-ft walkway width should be provided outside of the runway clearance. The walkway and the runway girder may be one continuous member, or for larger machines some cost savings may be attained by constructing the walkway of lighter steel or timber framing. Maximum travel speeds vary widely and may be up to around

140 ft/min. Turning radii, gradability, and operating clearances must be obtained from the manufacturer. More specific information on straddle carrier dimensions and requirements, as well as recommended dimensions for piers can be found on the Marine Travelift website (see Appendix 3).

Miscellaneous Boat-Handling Equipment

Other types of small-boat lifting and handling equipment include forklift trucks, bulkhead-mounted forklifts, and crane-type davit hoists, as well as hydraulic trailers and self-propelled transporters. Special forklift trucks have been developed for dry-stacking of small boats (typically power boats under around 25 to 30 ft). These lift trucks have extended forks that may be elevated to up to 40 ft above ground, and some have negative lift capability for extending their forks down to around 12 ft below grade to lift boats from the water. Such trucks are available with up to 11-metric ton capacity and corresponding front-axle loads of 117,000 lb. Maximum wheel and axle loads should be obtained from the manufacturer (see also Section 4.2 for discussion of forklift truck loads). Less common are bulkhead- or pier-mounted forklifts that rotate about their base at deck level. Capacities usually are limited to less than around 10 metric tons.

Crane-type hoists with fixed bases may be of various types; the most common probably is the jib boom type depicted in Fig. 10-51. The cantilevered boom can

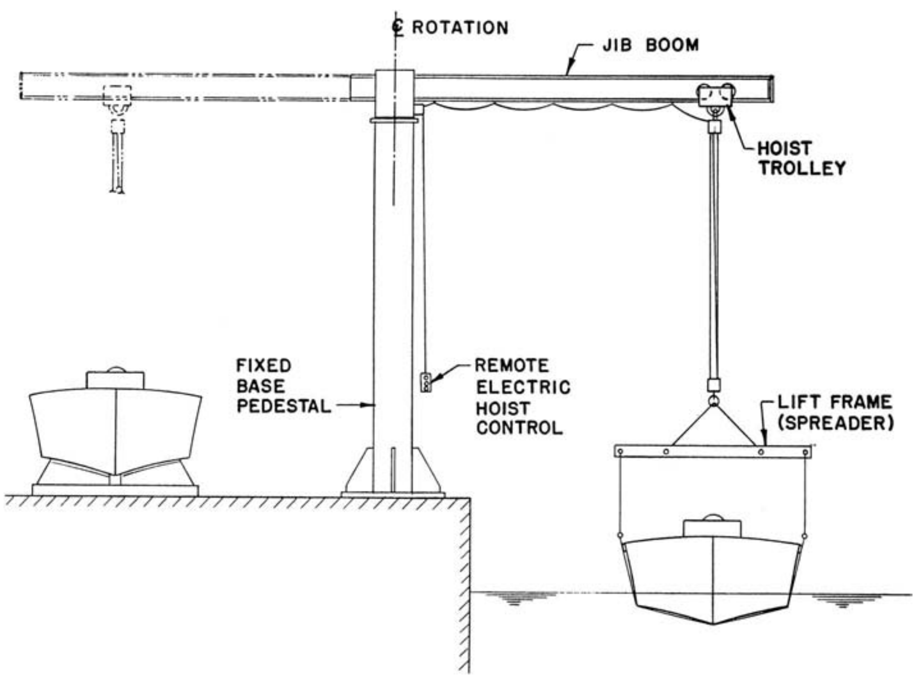


Fig. 10-51. Schematic of jib boom hoist for small craft

rotate on its pedestal base or column support and usually is equipped with a remote electric or manual hoist that can be fixed at its outboard end or mounted on a trolley that can move along the boom. These types of hoists usually are limited to 5 tons or less capacity, and less than a 10- to 15-ft outreach. Hydraulic pedestal-mounted cranes with fixed or telescoping booms, as designed for shipboard use, also may be used. They are available to much higher capacities but generally require substantial foundations and/or local strengthening of pier decks. Derrick-type cranes, consisting of a vertical post (mast) with a long boom hinged near its base and secured to the mast via a wire-rope topping lift, occasionally are seen. Hoisting is accomplished by a wire-rope winch, usually reeved in parts to reduce the line pull. The fixed-base pedestal and derrick-type hoists create large moments about their bases, which usually require substantial foundations or below-deck framing. The design of lifting equipment is beyond the scope of this book, and their installation should consider load and impact factors appropriate to the specific type of lifting device. Additional discussion of boatyard equipment can be found in Tobiasson and Kollemeyer (2000).

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CHAPTER 11

Rehabilitation, Maintenance, and Repair

Marine structures are located in a harsh environment and are often subjected to high loads from ships and deck cargoes and equipment. Without routine maintenance and periodic repairs, the service life of the structures can be significantly reduced. It is not uncommon for marine structures to have structural failures caused by deferred maintenance.

Inadequate maintenance, however, may only be part of the reason for structural failures. Often, the use of marine facilities changes over the life of the structure. The unit cost of moving materials by ship tends to decrease as capacity increases, providing an economic incentive for berthing larger ships and using larger cranes and associated loading/unloading equipment. For this reason, many facilities are upgraded and modernized to keep pace with developments in vessels and material handling technology. Facilities may also be converted to new uses as a result of changing demands. In most developed countries, there are few available undeveloped sites for new marine terminals; hence, there is a strong emphasis on restoring, upgrading, and/or converting existing facilities to new uses.

This chapter begins with a general discussion of important considerations in rehabilitating, reconstructing, and maintaining existing structures. Primary modes of deterioration are discussed in detail in the following section. A thorough understanding of the causes of the deterioration observed is critical to designing or selecting an appropriate repair method. A general approach to restoration and structural repairs is presented, and a separate section is devoted to pile and bulkhead restoration measures, which are among the most ubiquitous of port-structure problems. The application of protective coatings, claddings, and cathodic protection systems is reviewed. Finally, repair and replacement of fender systems and mooring hardware are presented.

11.1 Rehabilitation, Reconstruction, and Maintenance

Marine structures are subjected to harsh environmental and service conditions. A proper maintenance program involves constant vigilance and repair of damage and deterioration as they occur. This care is especially important for marine structures, in which structural failure or conditions that eventually render them unserviceable are the result of cumulative damage and gradual deterioration effects. Unfortunately, marine structures often suffer from neglect, reaching the stage at which major repairs or reconstruction is necessary. Good design and detailing practice and the proper selection and application of materials and protective systems greatly increase the life of a marine structure and reduce the chances of catastrophic failure. Certain structures may be deliberately overdesigned or designed with modern composite materials in order to lessen maintenance requirements over the life of the structure.

Some of the most common maintenance and repair problems in pier and wharf construction are

- Pile and bulkhead deterioration, especially within the splash zone and lower tidal zone;
- Superstructure deterioration, including corrosion, cracking, and spalling of concrete decks, beams, and pile caps;
- Crane-rail attachment problems resulting in spalling of grout and misalignment of rails;
- Fender-system deterioration and damage; and
- Mooring-hardware corrosion and overload damage.

The extent of the deterioration problem in a port structure is dependent on many factors, including the materials used in construction, the quality of the fabrication and installation, the severity of structure use, and the aggressiveness of the marine environment. Lifecycle management for port structures is covered in PIANC (1998, 2008); also see Section 12.5. Port and waterfront structure maintenance in general is addressed in DOD (2001a), ICE (1978), AAPA (1970), and Johnson (1965).

Rehabilitation versus Reconstruction

The decision to repair and rehabilitate versus reconstruct a marine facility often is an economic or political decision. New ship-berthing marine facilities cost millions of dollars to construct, and these large capital expenditures typically require approval from top-level management, government, or even congressional approval for U.S. military and federal government projects. For this reason, repair and rehabilitation are often selected over reconstruction, even if reconstruction appears justified by engineering lifecycle analysis. It is important to consider the lifecycle costs of the rehabilitated facility versus a newly constructed facility. The lifecycle cost can be evaluated on a first cost/present worth basis, with allowances for periodic maintenance costs for both the rehabilitated and new structure options. A newly constructed facility typically has lower maintenance costs compared to facilities incorporating aged materials.

If an existing marine structure is being upgraded in either dimensions or load capacity, which often occurs, then rehabilitating the structure to its former "like new" condition does not suffice and additional engineering analysis is required. Cargo-handling facilities, for example, usually need to be upgraded to meet the requirements of larger vessels, cargo-handling equipment, and methods. This upgrade often involves increases in the load-carrying capacity of the structure. On the other hand, if using a former cargo wharf or quay for office, residential, or recreational purposes, then the structure may still possess sufficient capacity, even if deteriorated, and the problem then involves prevention of further deterioration and aesthetic considerations, rather than increasing structural strength. The degree of deterioration and the nature of the current intended use thus become central to design of repair and rehabilitation. ICE (1986), BSCE (1984), and Wong (1983) contain useful papers on case histories in marine structure rehabilitation.

Upgrading and Modernization

Upgrading and modernizing an existing facility whenever feasible often produces initial cost savings over building a new facility or completely reconstructing an existing facility. The reuse of an existing facility also is likely to avoid many regulatory delays and environmental objections, which often occur in the permitting of a new facility. Modernization also can usually be carried out so as to minimize interference with ongoing activities, resulting in less facility downtime and a reduction in overall construction time. Upgrading usually is required as a result of changes in vessel and material handling technology, which typically means larger, deeper-draft vessels and heavier, faster cargo-handling equipment. Upgrading, modernization, and/or conversion of an existing facility usually involves any or all of the following requirements:

- Structural strengthening for both vertical and lateral loads,
- Dredging for maintenance or increased water depths and maneuvering area,
- Change in the overall layout and an increase in backland storage areas,
- Safety and pollution control improvements, and
- Provisions for new mechanical equipment.

If the existing structure can be restored to a "like new" condition, then other approaches may be taken to deal with increased loads. The use of modern high-energy-absorption resilient fender units can greatly reduce vessel berthing reaction forces, and the fender systems often can be designed around the lateral load capacity of the pier or wharf (see Section 5.6). Providing supplemental mooring structures or providing local reinforcing and/or means of load distribution at mooring hardware may help to handle increased mooring loads. Marginal wharf structures may have additional mooring hardware located along the backshore perimeter if the lead of the vessels' mooring lines is long enough. Strengthening of deck systems often involves the need to accommodate heavier wheel or crane loads, which sometimes can be achieved by the addition of topping slabs to distribute the concentrated loads, although this method may result in the loss of some uniform live-load capacity. Limiting the live-load storage capacity may be an acceptable trade-off in many instances. Modern methods of analysis sometimes may reveal certain reserve strength not counted upon in the original design. One should exercise caution in this regard, as certain building code requirements now are more conservative than earlier versions (Garlich 2001). More frequent application of dynamic analysis also has resulted in more conservative designs for new marine facilities. The high lateral rigidity of existing waterfront dock structures may preclude modification of these structures to meet modern seismic codes.

Upgrading of marine facilities often involves dredging of ship berths to allow for deeper draft vessels. Increased dredge depths at existing facilities, which were designed for shallower berths, may result in foundation-slope-stability problems for wharves and bulkheads and increased effective column lengths of foundation piles. Berth deepening may require the use of an underwater soldier pile/shutter panel type of retaining walls to hold back the under-wharf slope; however, the added cost of this type of system may be offset by using the soldier piles as part of the fender system. This method works quite well for submarine berths where the berthing loads are located well below water level and are transmitted into the under-wharf soils by the soldier pile/shutter panel system.

Facility layout requirements vary depending on the products being handled, but modern uses usually require the addition of space. Backland storage areas, paving, and roadways often require maintenance or upgrading (PIANC 1986). Older vintage general cargo terminals usually had transit sheds or warehouses built onto the wharves and piers themselves, whereas today's mobile equipment, larger cranes, and containerized cargo require larger and more open spaces than were once needed. Railroad spurs once were prevalent on piers and wharves, whereas today the number of tracks required, if any, generally is minimal.

Modern marine terminals must consider many safety-related and pollution control features that were not required in earlier times. Edges and openings typically must be guarded to current standards of the Occupational Safety and Health Administration, and often railings, ladders, and curbs on older waterfront structures need to be updated to comply with current requirements. Also, older marine facilities may not meet current fire protection and oil containment and collection standards. Allowing for future sea level rise may be challenging because of existing deck and backland elevations.

Provision for larger and heavier cargo-handling equipment is a frequent requirement when upgrading existing marine structures. Older gantry cranes often suffer from metal fatigue and should be inspected carefully if being reconditioned for new use (PIANC 1985). If longer reach or higher capacity cranes are desired, there may be requirements to modify the existing structure to accommodate a wider gauge crane-rail system, which may require additional piles and superstructure framing.

Rehabilitation Design Considerations

In approaching the problem of rehabilitating an existing waterfront structure, knowledge about the existing structure should be sought and carefully considered. If only a routine inspection (see Chapter 12) has been performed, then a more detailed investigation may be needed to gather data on the structure, such as

- Results of the material condition survey, including results of laboratory or nondestructive testing (NDT), such as cores and coupons, remaining material thickness measurements, and so on.
- Original construction documents (as-built drawings), maintenance and repair records, pile driving logs, service history, and so on.
- Results of soil borings and laboratory testing.
- Load-test results (if warranted).
- Review of vintage material specifications and design standards.
- Review of products and methods available for repairs and construction.

Review of all available information on the past history of the structure, including as-built construction drawings, pile driving records, maintenance and repair records, and the service history, can prove invaluable in assessing rehabilitation. Unfortunately, this information generally is hard to find, and when available on a piecemeal basis, it may be difficult to interpret. If construction drawings are not available, then the principal dimensions and member sizes must be ascertained in the field during the material condition survey. Even when record drawings are available, general dimensions and soundings should be verified during the survey. In some cases, especially if dredging, filling, or an increase in load capacity is planned, new soil borings may be required if no record documents are available and if there is any question as to the structure's stability and capacity.

In cases where borings may be unwarranted and/or inconclusive, and especially in structures where the substructure and foundation are not practically available to inspection, then full-scale load-testing may be the only way to verify capacity. The load test may consist of loading an entire area, as would be required in a retaining-type structure, and monitoring movements, pore pressures, and so forth; or it may be sufficient to selectively load-test individual piles. Load-testing is the most conclusive of the various ways to verify the integrity of an existing structure, but it is often costly and disruptive; so the engineer often must proceed with less-definitive information than load test results.

In evaluating the capacity of an existing structure, the engineer must be aware of the materials, methods, codes, and standards that were used at the time of the structure's original construction (Garlich 2001). For this purpose, maintaining a file of outdated codes and handbooks may prove useful. The designer also may be confronted with a serious dilemma in analyzing an older structure under current standards, a problem that often is not easily resolved. In this instance, the designer's judgment must be applied on an individual-case basis as to whether the past service record constitutes an effective load test, or whether strengthening to current standards is warranted. A case history example is provided in Gaythwaite and Carchedi (1984).

Finally, in approaching the rehabilitation problem, the designer must be fully familiar with the materials and methods currently available for restoration and marine construction in general (see Section 3.5). This step may not be as simple as it sounds as there are a wide variety of products being marketed as protective coatings and patch-and-fill repair-type compounds, for example, that may require some investigation to verify the manufacturers' claims and applicability to the repair at hand. Specifying particular products requires a knowledge of the causes of the deterioration, engineering properties of the repair materials, and satisfactory documented performance records in similar repair applications in support of manufacturers' claims.

11.2 Deterioration Modes

Marine structures are subject to relatively rapid rates of deterioration caused by environmental forces, such as waves, currents, tides, extreme water levels, and ice, and mechanisms such as corrosion, physical/chemical attacks, biodeterioration, general wear, abrasion, fatigue, and damage caused by vessel impact and overloads. Structures also may become unserviceable because of the movement of bottom materials (i.e., through scour and siltation effects). Rapid deterioration rates also may be attributed to poor design, workmanship, and quality of materials. Environmental factors are discussed in Section 3.4 and their effects on material selection in Section 3.5. This section is intended to provide a general overview of deterioration mechanisms. A thorough understanding of the causes of deterioration is critical to the appropriate selection and specification of repair materials and methods. An overview of inspection, maintenance, and repair of structures subject to damage and degradation caused by the marine environment can be found in PIANC (2004). The site-specific physical and chemical properties of seawater often play an important role in deterioration rates. A more thorough overview of the effects of the marine environment on structural design can be found in Gaythwaite (1981). The general properties of various materials for marine use are described in Whiteneck and Hockney (1989) and USACE (1983).

Principal modes of deterioration are generally specific to material type, and examples of their primary effects are as follows:

- Corrosion of metals, including concrete-reinforcing and prestressing steel;
- Biodeterioration, such as the decay of timber and attack by marine organisms;
- Physicochemical processes, such as freeze–thaw damage, alkali–aggregate reaction (AAR), delayed ettringite formation, and chemical attack of concrete; and
- Mechanical damage, such as abrasion, wear, overloading, and breakage, which affects all material types.

The overall long-term durability of a structure depends upon local environmental conditions, the material of construction, and severity of use. The rates and severity of deterioration typically exhibit a vertical zonation, as illustrated in Fig. 3-18, in which tide range and extreme water levels are important factors. Seawater properties such as temperature, salinity, dissolved oxygen, pH, and the presence of pollutants, as well as currents and exposure to wave action, are important environmental factors. The chemical state of the ocean is notably affected by the concentration of the major ions, the acidity (as measured by pH), and the oxidation–reduction (redox) potential. Seawater is slightly alkaline with a pH typically around 8. Deterioration also may be localized to vulnerable and heavy-use areas, or to areas where incompatible materials are mixed, or to areas of poor construction quality. The basic modes of deterioration are discussed in the following subsections.

Corrosion of Steel

Corrosion is one of the leading causes of deterioration of steel, steel-reinforced concrete, and steel-fastened timber waterfront structures. Many of the cracks, spalls, and delaminations found in concrete are caused by corrosion of the reinforcing steel. Corrosion is an electrochemical process whereby atoms of the metal lose electrons, thus becoming positively charged (oxidation). The free electrons then combine with atoms of an adjacent area or a surrounding substance (reduction). The electric current or migration of electrons from an anodic to a cathodic area may be caused or accelerated by various conditions that define the type and rate of corrosion found on a given structure. Areas on the same piece of steel may become anodic and cathodic with respect to one another, thus creating a local corrosion cell. The seawater acts as a conducting medium or *electrolyte*. The general chemical (half-cell) reactions are

$$\text{At Anode: } Fe = Fe^{++} + 2e \tag{11-1}$$

$$\text{At Cathode: } H_2O + 1/2O_2 + 2e = 2OH^- \tag{11-2}$$

Combining the above equations results in the formation of ferrous hydroxide, $Fe(OH)_2$ and possibly other iron minerals, such as goethite, $FeO(OH)$ or hematite, Fe_2O_3. When the localized areas of potential difference are closely spaced, the corrosion becomes somewhat uniform as the corrosion cells shift position and general corrosion results. When the cathodic and anodic areas are more widely separated such as between the tidal zone and low water, a larger overall corrosion cell may be set up with more concentrated local corrosion in the anodic zone.

An in-depth explanation of corrosion processes is beyond the scope of this text. More rigorous treatment of marine corrosion can be found in Dismuke et al. (1981),

Fig. 11-1. Severe corrosion of a double bitt

Source: Photo courtesy of Appledore Marine Engineering, LLC

Laque (1975), and Uhlig (1948), and the NACE *Corrosion Engineers Reference Book* (Baboian 2002) contains numerous tables and useful corrosion reference data.

Principal types of corrosion normally encountered on waterfront structures include *general* and *pitting* corrosion, manifested by scaling, pitting, or general wastage of large areas, such as is evident in Fig. 11-1, caused by reaction of the metal with its surrounding medium or with cathodic areas of the same member. *Crevice* corrosion, found in isolated areas, is caused by localized oxygen depletion, such as that found under bolt heads and at discontinuous welds and joints. There is also *galvanic* corrosion caused by different electric potentials between dissimilar metals immersed in seawater. Table 11-1 shows the voltage potential, relative to a standard calomel electrode (SCE), for selected materials immersed in quiescent seawater. The greater the potential difference, the more aggressive the corrosion; more negative potential is anodic. *Corrosion fatigue* and *stress corrosion cracking*, related to cyclic loading and high tensile stresses, respectively, may occur in high-strength steels in particular.

The average rate of metal loss for mild steel in quiescent seawater is about 5 mil (0.005 in.)/year. Localized pitting and crevice corrosion may proceed at several times this rate, however. Tidal currents (water velocity) greatly increase corrosion, on the order of three times the average rate for a 1 1/2-knot current to almost five times the rate for a 4-knot current. Turbulence and mixing accelerate corrosion rates by increasing the oxygen content and dislocating corrosion by-products, thus exposing a new substrate. Eroded sand or suspended sediments result in *erosion corrosion*.

Fig. 11-2 illustrates the typical vertical distribution of representative corrosion rates on sheet-pile bulkheads and piles in temperate climates. It shows highest areas

Table 11-1. Galvanic Series of Metals in Quiet Seawater with Emphasis on Structural Steel

		Metal	Voltage Potential (SCE)	
(−) Anodic		Magnesium alloy	−1.6	
			−1.4	
			−1.2	
		Zinc		
		Aluminum alloy	−1.0	
		Galvanized iron		
			−0.8	
		Mild steel (clean)	−0.6	Current flow
	Lead	Cast iron		
	Stainless steel (active)	Mild steel (corroded)	−0.4	
		Mild steel (in concrete)	−0.2	
	Brass	Mill scale on steel		
	Copper		0.0	
	Bronze	Stainless steel (passive)		
			+0.2	
	Graphite			
			+0.4	
(+) Cathodic				

Notes: Relative position of metals on voltage scale are approximate for average conditions but may vary considerably for given metal depending upon alloy content and other physiochemical factors. Ranges of potential are shown for mild steel in order to emphasize the possibility of galvanic action on the same metal.

of loss in the splash zone and immediately at and below mean low water (MLW). Such typical profiles cannot necessarily be applied to all structures because the vertical distribution, which arises primarily from zones of differential aeration on the metal surface, may vary markedly under differing site conditions. For example, in some harbors with relatively large tide ranges, experience seems to indicate a much less pronounced spike in the splash zone with greatest attack just below MLW and, sometimes, again just above the mudline, especially at shallow-water sites. Increased corrosion rates may be found at the mudline of structures exposed to wave and current action caused by erosion corrosion. Many sites are subject to accelerated low water corrosion (ALWC) caused primarily by microbiologically induced corrosion (MIC), with corrosion rates within the low water zone several times the average rate. ALWC most commonly occurs as a horizontal band around low water but may also be found in patches and extend down to the mudline (PIANC 2005). The band of ALWC typically begins around the lowest astronomic tide (LAT) level and extends to 0.5 m above into the low water zone. It often appears as a lightly adherent orange

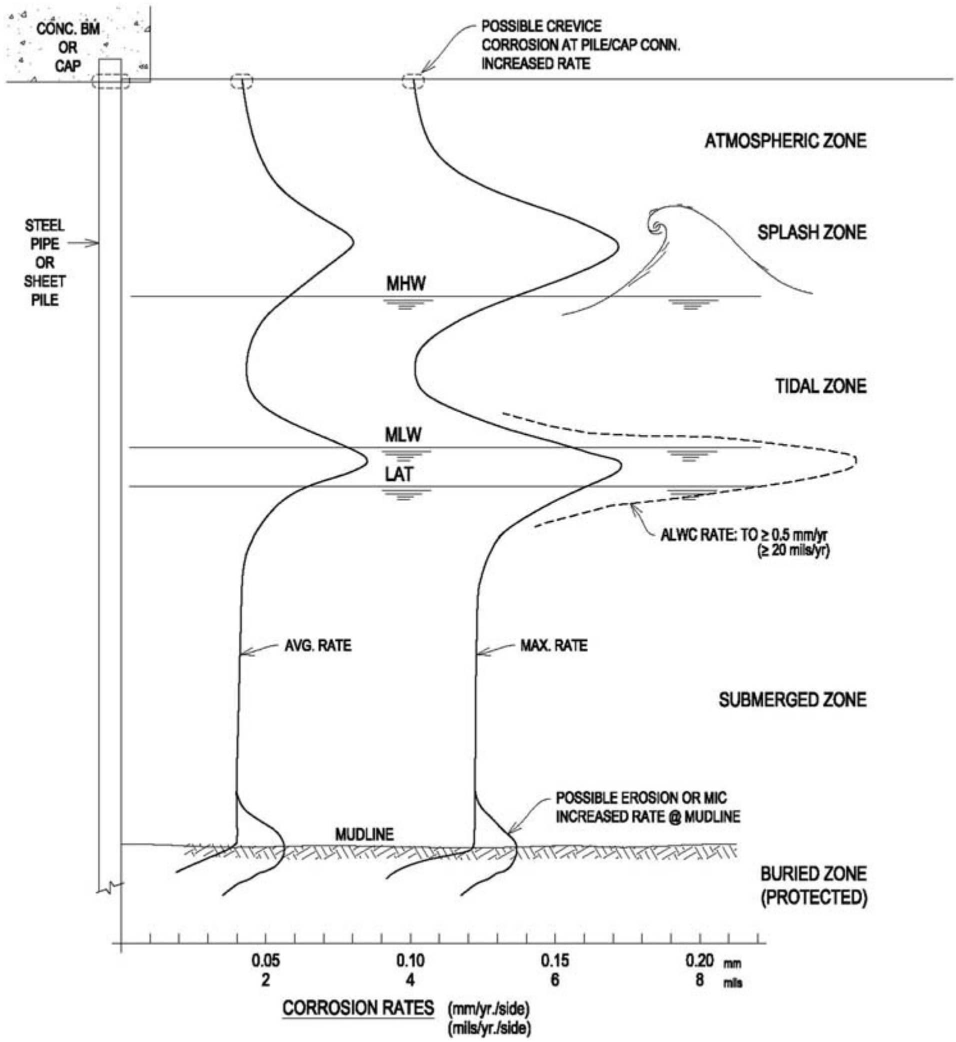

Fig. 11-2. Vertical profile of representative corrosion rates for steel piles and sheet piles in temperate climate

patch or zone $[Fe(OH)_2]$ with an underlying black sludge of iron sulfides (FeS) also revealed by the smell of hydrogen sulfide (H_2S), which when scraped clean reveals a shiny steel surface. ALWC occurs because of the presence of sulfates in the seawater, which are converted to H_2S by sulfate-reducing bacteria (SRBs), which cause direct corrosion of steel surfaces and serve as a source of nourishment for sulfate-oxidizing bacteria (SOBs), which in turn convert H_2S to sulfuric acid (H_2SO_4), thus promoting a continuous electrolytic corrosion of the steel surface. Additional description and strategies for management of ALWC in maritime structures can be found in CIRIA (2005).

Corrosion rates for sheet piling are approximately doubled when exposed to seawater on both sides versus one side filled. Studies of sheet-pile bulkheads at several U.S. naval stations (Ayers and Stokes 1961) revealed maximum corrosion rates on the order of 8 mil/year in temperate climates, up to 19 mil/year in subtropical climates. Maximum rates were on the order of twice the average rates. Once perforated, corrosion may proceed at the same rate on both sides of sheet piles. Local perforations may render a bulkhead unserviceable because of leakage of fill long before general wastage reduces its structural strength to unacceptable levels (Buslov 1983). Splash zone rates on offshore structures of 55 mil/year have been reported in the Gulf of Mexico (Creamer 1970), although maximum splash zone rates on offshore structures are more typically in the range of 25 to 40 mil/year. Stray electric currents may be a source of rapid corrosion rates, especially at shipyards where improper grounding of welding equipment or electric utility systems may result in localized electric currents and related corrosion. Faulty impressed-current cathodic protection systems, high-voltage cable crossings, berthed vessels, or other extraneous sources also may contribute to high localized corrosion rates caused by stray electric currents. The handbook by Dismuke et al. (1981) is especially useful to the practicing waterfront engineer in providing detailed information on expected rates, steel selection, and protection of steel piles, and DOD (1992) provides general information on corrosion control.

An important point about corrosion that is especially relevant to repair work concerns the nature of galvanic action on new and corroded steel specimens. Because corroded steel is cathodic to new, clean steel (Table 11-1), a new patch welded to an existing pile, for example, corrodes at a higher rate than the pile. When a steel pile is repaired by a partial-length concrete encasement that does not extend far enough below MLW, the unprotected portion of pile below the concrete encasement may become anodic to the encased pile and corrodes at an accelerated rate, as shown in Fig. 11-3.

Biodeterioration of Wood

Attack by marine organisms (borers) is an ever-present threat to timber supported marine structures in tropical and temperate climates. Two types of borers are prominent: the "wood gribble" and other members of the *Limnoria* family, which are crustaceans, and the "*Teredo*," which is a mollusk related to the clam, although it is wormlike in appearance. Fig. 11-4 illustrates the characteristic patterns of damage inflicted by these two types of borers. The *Limnoria* nibble away shallow furrows at the surface and are responsible for the narrowed like the middle of an hourglass appearance of timber piles often observed in the submerged zone. *Limnoria* damage can reduce a pile diameter by as much as 2 in. per year (Chellis 1961). Another common area of *Limnoria* damage is at the ends of dimensioned lumber, where the preservative impregnation is marginal or may have been totally eliminated if the end was trimmed during construction. The protection against marine borers provided by

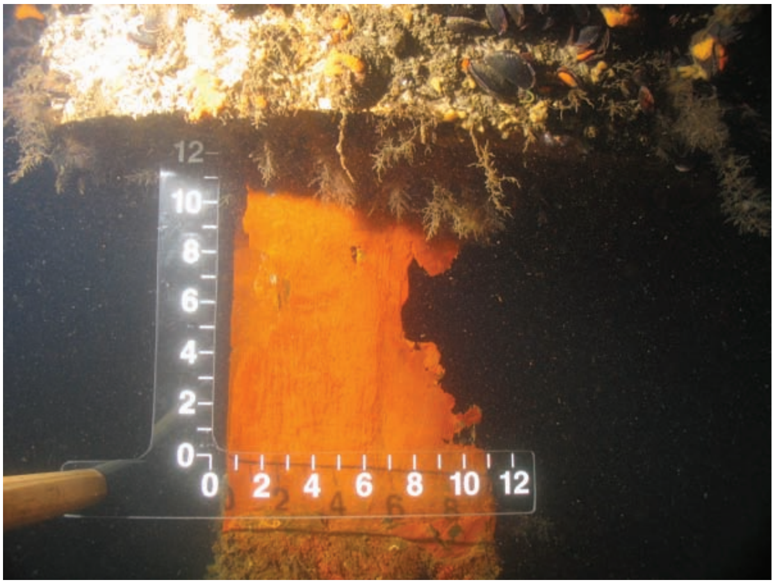

Fig. 11-3. Severe galvanic corrosion of steel pile below concrete jacket

Source: Photo courtesy of Appledore Marine Engineering, LLC

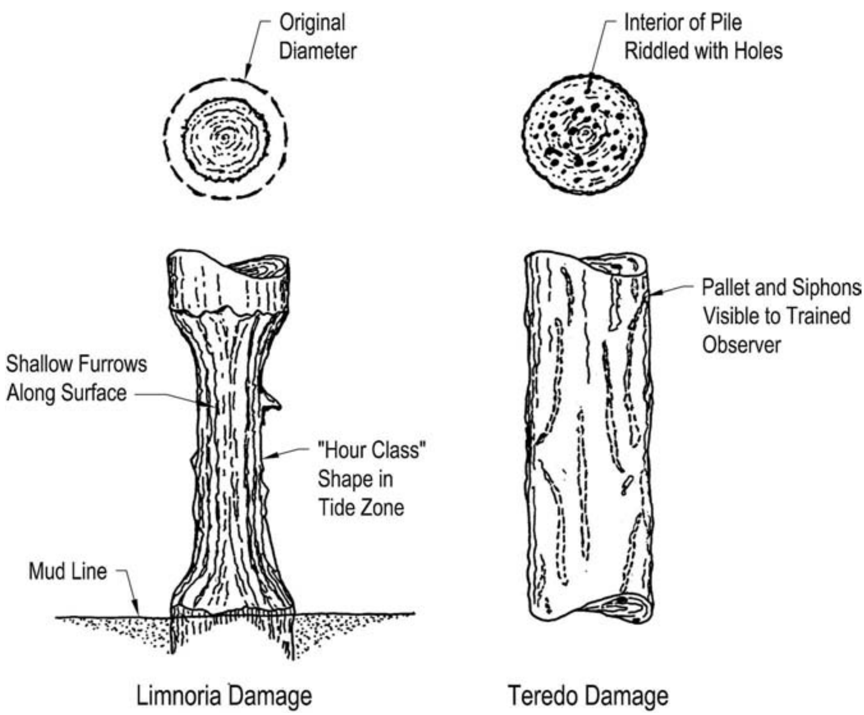

Fig. 11-4. Marine borer damage of wood piles

Source: Gaythwaite and Carchedi (1984)

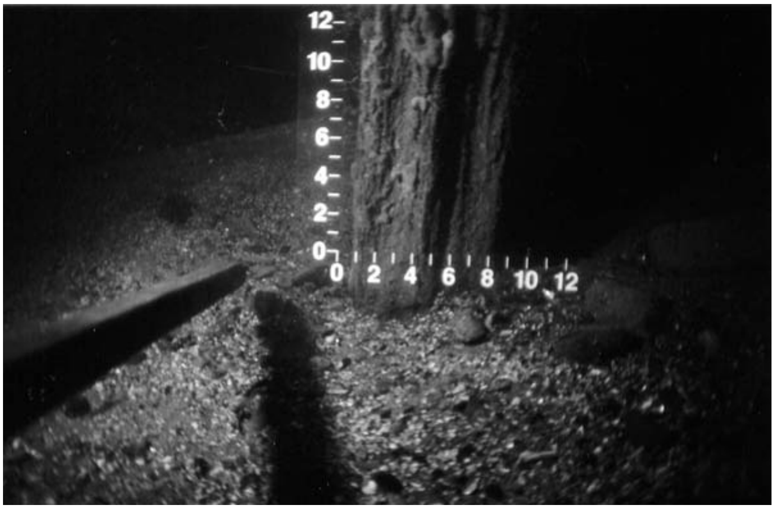

Fig. 11-5. Marine borer damage to wood pile near mudline

Source: Photo courtesy of Appledore Marine Engineering, LLC

creosote treatment may eventually leach out over time, allowing the infestation of the timber, as shown in Fig. 11-5. *Teredo* damage is less obvious because the animal grows inside the wood as it tunnels along, leaving the typically untreated inside of the pile riddled with holes. The adult *Teredo* actually may burrow to above mean high water. *Teredo* damage can reduce interior pile area rapidly. All marine borers exhibit some degree of sensitivity to seawater properties, including temperature, salinity, pH, currents, and the presence of pollutants. Optimum temperature ranges, depending upon the species, usually range from around 40° to 70°F, although certain species of *Limnoria* can survive in almost freezing water. The pH generally must be within the range of 7.5 to 8.5 for all species. The *Teredo* can tolerate salinities as low as eight parts per thousand (8 ppt), but *Limnoria* becomes inactive at salinities below 16 ppt to 12 ppt. Both *Limnoria* and the *Teredo* are unlikely to attack when currents approach and exceed around 2 knots, and all species appear to have low resistance to general harbor pollution. Lopez-Anido et al. (2004) report on *Limnoria* and *Teredo* activity along the coast of Maine, demonstrating that marine borer activity can be pervasive in relatively cold water climates. Other crustacean borers include the *Sphaeroma* and *Chelura* families. The *Pholad* family of molluscan borers has been known to penetrate rock and poor-quality concrete. The genus *Lithophaga*, or "date mussels," and certain boring sponges chemically attack limestone and carbonate rocks and have caused severe damage to concrete piles in the Arabian Gulf (Jadkowski and Wiltsie 1985). Means of classifying and quantifying marine borer damage can be found in NAVDOCKS (1965).

Various forms of marine bacteria may play an important role in promoting biodeterioration, fouling, and corrosion. Sulfate-reducing anaerobic bacteria may produce hydrogen sulfide (H_2S) within bottom sediments and under the blisters of protective coatings or the bases of sessile fouling organisms. H_2S may affect the corrosion history of steel structures as described previously and of steel fastenings in wood. Other forms of bacteria may even affect buried concrete or promote the breakdown of oil-based wood preservatives.

Timber structures often suffer from rot of the superstructure, deck, and stringers largely because of stagnant freshwater. Bacteria, various types of fungi, and insects, sometimes including wharf-boring beetles (*Nacerda*), often are evidenced in older, neglected timber structures. Rot is insidious and may be difficult to detect, especially in stringers and pile caps, where the only evidence may be a surface coating of fuzz or slime or the characteristic brown or white powderlike residue of dry rot.

The most common and destructive type of decay organisms are fungi responsible for both brown and white dry rot and other forms of rot, ranging from surface discoloration to total destruction of the wood (Highly and Scheffer 1989). Such advanced stages usually are evidenced by the presence of fruiting bodies or a threadlike mycelium. Certain forms that cause only discoloration of the wood may not affect its static structural strength but can affect its impact resistance dramatically, with obvious serious implications for waterfront structures. Additional information on evaluation of deterioration of wood structures can be found in ASCE (1982).

Timber deck systems must be given careful scrutiny, and provisions must be made for their future protection if they are being planned for reuse. Fig. 11-6 illustrates areas of a timber pier where decay is most commonly found. Corroded bolts within the wood often give the wood surface a black to blue discoloration. A common sight along the waterfront is deteriorated top butt ends of fender piles that were not protected with some form of cladding.

Physical and Chemical Deterioration of Concrete

Concrete is subject to physical damage by abrasion and freeze–thaw cycles and chemical attack by chlorides, carbon dioxide (CO_2), sulfates, and reactive alkali aggregates (AAR). Concrete reinforcing, both plain and prestressed, may be subject to electrochemical attack as well as that described previously for steel structures. Corrosion of reinforcing steel is a common and pervasive cause of concrete deterioration. Expansion of steel corrosion products initiates stresses in the concrete. In temperate and cold climates, concrete is especially vulnerable in the tidal zone, where, with each tidal cycle, the concrete goes through a freeze–thaw cycle during the winter months. This effect is caused by the approximately 9% expansion of water upon freezing and may be exacerbated by the abrasive action of ice rising and falling with the tide or borne by currents. Frost resistance of concrete is related to the pore structure of the matrix and so air-entraining agents are normally added in cold regions to allow space for trapped moisture to expand. Concrete permeability and cracking are important factors in the initiation and progression of most forms of concrete deterioration. A powdery residue known as “efflorescence” is often seen on

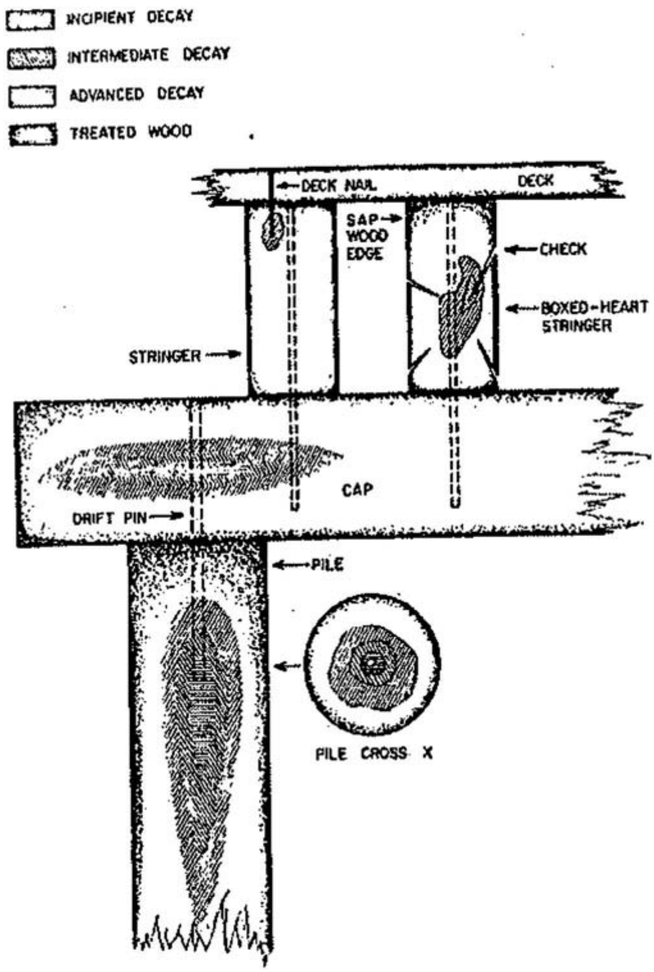

Fig. 11-6. Schematic illustration of typical locations of decay damage on timber pile bent

Source: ASCE (1982); copyright ASCE

the underside of deck slabs and sometimes beams as well, indicating that water has passed through the concrete and washed out some of the cement paste, resulting in a residue of calcium hydroxide [$Ca(OH)_2$] crystals. Where cracks are present, "stalagtites" of $Ca(OH)_2$ and other calcareous by-products may also be seen hanging from the bottom of the slab or beams.

Chloride penetration is perhaps the most severe and pervasive form of chemical attack and occurs mostly in the splash and tide zones, where water and oxygen contents are optimal for corrosion of reinforcing steel. Corrosion of reinforcing steel is initiated when chloride ions and oxygen penetrate the concrete cover and destroy the passive layer on the reinforcing. It is a complex electrochemical process (Bazant

1979), resulting in depassivation of the steel when the pH value of the concrete cover (normally around 13.0) is lowered below about 11.5. The pH of seawater normally is within the range of 8.0 and 8.5. Chloride ion penetration as a measure of concrete permeability is measured in coulombs (C), a unit of electrical charge, under the ASTM C1202 rapid chloride permeability test. High permeability is >4,000 C, and low permeability is <1,000 C. Electrical resistivity methods offer alternative and perhaps more expeditious means of measuring concrete permeability (Laysii et al. 2015).

Carbon dioxide penetrating the concrete cover causes the formation of calcium carbonate ($CaCO_3$), called *carbonation*, which reduces the concrete alkalinity and also results in the depassivation of the reinforcing steel. Carbon dioxide cannot penetrate saturated concrete, however, and is primarily restricted to deterioration in the atmospheric zone.

Sulfate attack of seawater constituents on the calcium hydroxide [$Ca(OH)_2$] and/or tricalcium aluminate (C_3A) of the hardened cement paste can result in softening and degradation of the concrete as well as expansion and leaching caused by the formation of ettringite and gypsum. Sulfate attack is not usually a significant problem in good-quality concrete.

If there is widespread softening of the surficial concrete and map pattern cracking (ACI 1998a), then the concrete may have alkali–silica reaction (ASR), a specific form of AAR, deterioration (Farny and Kosmatka 1997). ASR deterioration has been identified in bridges and waterfront structures around the United States, though it is sometimes mistakenly identified as delayed ettringite formation (DEF), which has similar symptoms but is associated with precast concrete that has been heat-cured (Erlin 1999). Petrographic examination of the concrete is used to determine the cause of chemical deterioration. The uranyl acetate fluorescence method is an economical technique to test for ASR (Emmons 1993).

Improper design, application, and workmanship, as well as erosion and overloading damage, serve to accelerate the physicochemical deterioration processes. Note the severe scaling and spalling of the concrete deck in Fig. 11-7 adjacent to the expansion joints and rail pockets. The use of bituminous concrete for patchwork repairs probably has accelerated the degradation process through the capture and condensation of moisture at the concrete surface. Concrete sometimes is used as cladding to protect steel piles in the splash and tidal zones, although this protection is done where concrete itself suffers most in freezing climates. As can be seen in Fig. 11-8, unless the concrete is properly installed and quality controlled, the concrete jackets or shafts themselves may represent the biggest maintenance problem. More general discussion of concrete deterioration and quality can be found in ACI (2014a, b), and ACI (1988, 1982, 1980) evaluate the actual performance of concrete in seawater.

Mechanical Damage and Deterioration

Waterfront structures, in particular fender systems (Padron and Han 1983), are subject to mechanical damage caused by accidental vessel impact, such as the pile

Fig. 11-7. Degradation of concrete wharf deck at expansion joint and rail pockets. Bituminous patches have likely accelerated the deterioration

Source: Photo courtesy of the Maguire Group, Inc.

Fig. 11-8. Deterioration of concrete pile shafts in the tidal zone. Piles to the left have been partially restored by the use of concrete-filled fabric forms. Installation of contemporary resilient fender units helps reduce berthing impact loads

Source: Photo courtesy of the Maguire Group, Inc.

Fig. 11-9. Impact damage to previously repaired concrete pile

Source: Photo courtesy of Appledore Marine Engineering, LLC

shown in Fig. 11-9 and the cumulative effects of frequent berthing. Timber structures are especially susceptible to abrasion and wear deterioration caused by ice and vehicle traffic as well as vessels. Although wave action on an inner harbor structure may not exert significant forces to the structure, over time, cumulative erosion effects caused by entrained sand, debris, and so on can have a pronounced effect at exposed sites. During severe storms and when extreme water levels allow waves to penetrate

Fig. 11-10. Failed concrete deck because of bollard pullout likely caused by poor design of deck and anchorage and deterioration of concrete combined with high mooring line load

Source: Photo courtesy of Appledore Marine Engineering, LLC

where they normally would not reach, mechanical damage caused by thrown debris and plucking of riprap slope protection may result.

Waterfront structures often are subjected to overloads caused by operations. This is especially true for an older structure and may be a major factor in its requiring rehabilitation, as current trends toward higher capacity cargo-handling equipment, larger vessels, and stacking of palletized and containerized cargo may subject the structure to loads well beyond the original design capacity. Fig. 11-10 shows the failure of a concrete deck at an overloaded bollard. This failure was also likely caused by faulty design and construction, as well as an excessive bollard load. The effects of poor design and construction may not become apparent for many years or until a design load condition is realized. Waterfront structures may also be overloaded by the occasional use of equipment, including mobile cranes and large forklifts, used for repair, construction, or handling of unusually heavy cargoes. Frequent breakage of fender piles, for example, indicates the need to upgrade the fender system. Signs of overload usually are readily detected; permanent deformations, settlements, localized damage such as from dropping heavy weights on deck, misalignment of members or the entire structure, failure of bracing or secondary members, and so on all indicate overload conditions. Fig. 11-11 shows a pavement failure caused by loss of fill through a severely corroded steel sheet pile bulkhead. This failure was likely

Fig. 11-11. Failed pavement and washout of fill behind steel sheet pile bulkhead caused by loss of fill through severely corroded bulkhead

Source: Photo courtesy of Appledore Marine Engineering, LLC

preceded by smaller pavement slumps and sinkholes at the inside face of the bulkhead, a telltale sign presaging the larger failure.

Stone Masonry Construction

Stone wall deterioration does not fall neatly within the previously discussed categories. Stone masonry is especially prevalent in older structures, partially because concrete durability in the tidal zone, particularly when exposed to freeze–thaw, was historically poor, and stone or stone veneer provided much better durability. For this reason, many stone masonry marine structures are found in New England (Vine and Rosen 1992) and in Europe.

Stone wall construction can be categorized by the shape and fitting of the individual blocks, ranging from irregular to rough-squared *rubblestone* to "dressed" and finished *ashlar* masonry. Walls can also be categorized by the pattern of joints known as "bond." A horizontal row of stone is called a *course*, and a course with stones of equal "rise" or height are referred to as *ranged*. Quality wall construction should have stone blocks placed in "stretcher" and "header" fashion with alternate stretchers extending back into the fill behind the wall. Many older walls were laid "dry," without mortar, and hence often exhibit "sinkholes" behind their top surfaces caused by loss of backfill through the open joints. Smaller stones called "pins" or "chinking" are often placed within the open joints as fill or spacers. Provisions for drainage from behind the wall must be made for walls that do not have open joints in their lower courses. Understanding the basics of stone masonry wall construction is key to evaluating stone wall problems. The inspection and maintenance of stone masonry structures is discussed in detail in Bray and Tatham (1991) and DOD (2001d), and McElroy and Lienhart (1993) cover stone durability and testing. The following checklist outlines commonly occurring problems with masonry walls:

- Settlements—uniform and differential
- Sinkholes, loss of backfill, soil deposits in front of open joints
- Large voids/open joints, loss of chinking stones, joint movement
- Missing or displaced blocks
- Stone size range and placement pattern—too small and poorly fitting stone, angular versus rounded
- Movement and misalignment—horizontal alignment
- Material degradation—cracking and surficial spalling caused by fire, freeze–thaw, and impact; poor stone quality; weathering caused by chemical attack, running water, and so on; spalling; and deterioration of mortared joints
- Scour and erosion—undermined or exposed footings
- Poor construction—past repairs, poor soils, poor drainage
- Damage caused by impact, ground movement, water pressure, and so on
- Deterioration of mechanical anchors—corrosion of dowels, cramp irons, and so on

Miscellaneous Materials Degradation

Composite materials may suffer degradation associated with the quality of their manufacture, exposure to various chemicals and especially to ultraviolet (UV) radiation. Fiber-reinforced plastic (FRP) pile jackets are susceptible to abrasion damage in certain exposures. Because of the wide variety of possible compositions, it is beyond the scope of this text to attempt to elaborate on all of the possibilities, although composites are mostly free of many of the problems affecting steel, concrete, and wood as described in the foregoing discussion. Some problems in the degradation of protective coatings and claddings are introduced in Section 11.5. Bridge bearings, seismic isolation bearings, and other mechanical linkages may have their own specific deterioration issues that are not addressed in this text and generally require specialized experience.

11.3 General Repair Methods

A thorough understanding of the causes of deterioration is critical to appropriate selection and specification of repair materials and methods. Any proposed method of structural repair should consider amelioration of the conditions that led to the original deterioration. For example, if improper deck drainage led to the decay of timber framing elements, then the drainage situation should be remedied before the rotted timber is replaced. If a fender system is suffering continual damage, then it should be replaced with a more resilient system with a higher energy-absorption capability. If the deterioration was a result of faulty workmanship or materials, replacement in kind with properly installed, good-quality materials should be sufficient. Repair methods also must consider relative cost, downtime and disruption of operations, longevity of the repairs, access of personnel and equipment, and structural integrity of the completed work. Repair methods vary with materials and member types, as well as location with respect to tidewater levels. The general approach to design of repairs should consider the following:

- Repair methodology and techniques,
- Removal and preparation of existing items being repaired,
- Selection of repair materials,
- Protective systems, and
- Strengthening techniques as required.

A general overview and design guidance for the inspection, maintenance, and repair of maritime structures is provided by PIANC (2004).

Concrete

Concrete is subject to various forms of physical and chemical attack, as described in the previous section. General design guidance for concrete repairs is provided by ACI (2014a), ACI/ICRI (2013), DOD (2001a), and USACE (1983). Guidance for the selection of materials for concrete repairs can be found in ACI (2006), and Alexander et al. (2006) have edited the many papers presented at an important specialty conference on concrete repair and rehabilitation. Specifications for concrete repairs must be based upon a correct diagnosis of the original cause of deterioration and must provide specific guidance for the implementation of repairs, materials to be used, and level of workmanship required (Snover et al. 2011).

In freezing climates, concrete is especially vulnerable in the tidal zone, where with each tidal cycle, the concrete goes through a freeze–thaw cycle during the winter months. The best protection against this form of attack is to provide the most impermeable concrete possible, as described in Section 3.5. The use of air entrainment is essential in freeze–thaw climates.

Corrosion of reinforcing steel, associated with penetration of the concrete cover by chloride and oxygen ions, is a common cause of concrete degradation, especially within the tidal and splash zones. The use of dense, impermeable concrete and an adequate cover is again the best protection against this type of degradation.

Concrete problems generally manifest themselves in cracks, spalls, and surface erosion. Cracks that are not subject to further movements, such as those caused by initial support settlements or volume change of the concrete during curing, may be sealed by pressure injection with epoxy, urethane, or methyl methacrylate, provided that the in-place concrete is sound. These compounds have excellent bond strength and essentially can restore the full strength of the concrete across the crack. Cracks as small as 2 mil (0.002 in.) to as large as 250 mil have been sealed by the pressure-injection process. Crack widths of less than approximately 6 mil (0.006 in.) generally may be considered small enough not to allow moisture and chlorides to penetrate to the reinforcing steel. Cracks that are subject to movements, such as those caused by temperature change, differential settlement, or structure deflections, may be routed and sealed with elastomeric sealants, provided that the cracks do not affect the overall structural integrity. The use of rigid-set injection materials to repair cracks subject to movement only causes new cracks to appear or the injected crack to reopen.

Spalls or localized pockets where the concrete has been chipped or eroded away often are patched with epoxy or polymer-modified mortars and cements. The choice of the appropriate mortar type and mix depends upon the extent and depth of the spall and whether it is on a horizontal, vertical, or overhead surface. The repair of cracks and partial-depth spalls is covered in greater detail in DOD (2001b). Where reinforcing steel is exposed, it must be properly cleaned, adjacent chloride-contaminated concrete must be removed (including under the bars), and a bonding agent must be applied just before repair mortar application (ICRI 2008). In some cases, it may be appropriate to consider the use of cathodic protection (CP) for the reinforcing steel (Sgouros et al. 1996, Kessler et al. 2002, Ball and Whitmore 2009). Cathodic protection of reinforcing steel may be particularly helpful in reducing the *halo* effect, wherein new steel adjoining existing steel in a localized repair may otherwise become anodic to the existing steel. When reinforcing corrosion has caused a reduction in rebar diameter or wire-strand section loss, analysis should be performed to decide if structural repairs are needed. Because of the spiral construction, large surface area, and relatively small diameter of the wires in prestressing strands, only a few years of corrosion may cause failure of prestressed concrete members.

To develop a lasting concrete repair, all deteriorated concrete must be chipped away and cleaned so that the mortar or patching compound adheres to a clean and sound substrate. If there is widespread softening of the surficial concrete and map-pattern cracking, then the concrete may have a form of AAR, such as alkali–silica reaction (ASR) deterioration (ACI 1998a, Farny and Kosmatka 1997). If tests of concrete samples do indicate the presence of ASR in the concrete, then the concrete may be swelling as it absorbs moisture and surface concrete repairs may not last long before debonding occurs. The use of lithium, either as an admixture in new concrete, or as a surface coating on existing concrete, has shown some promise

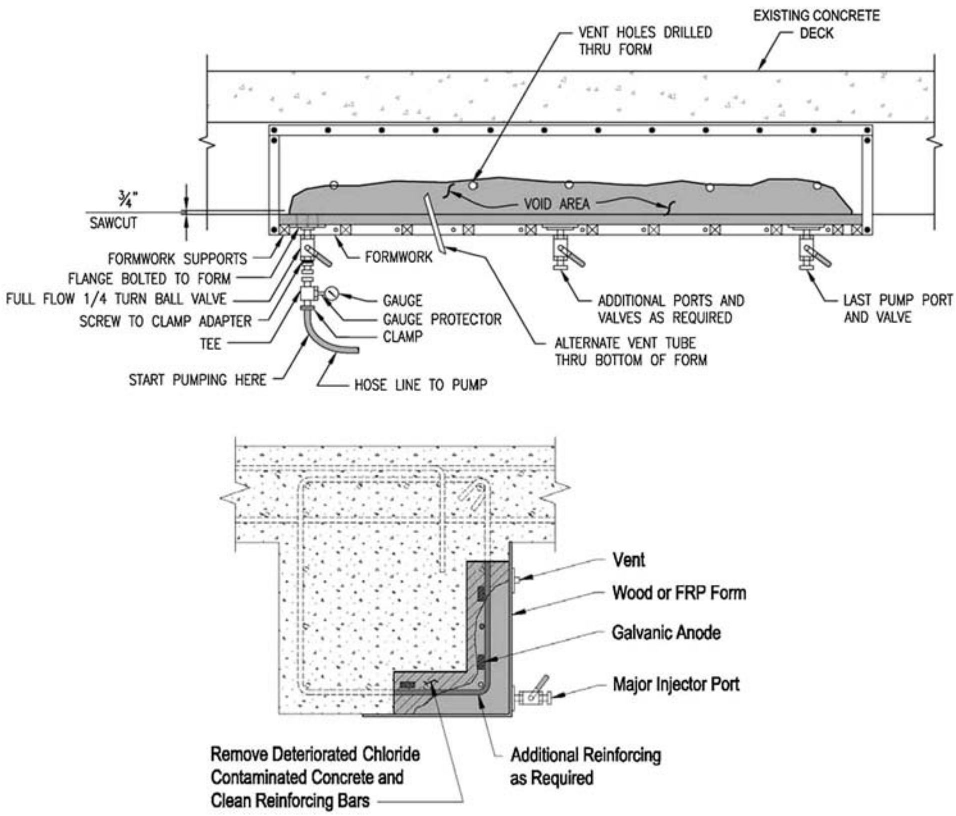

Fig. 11-12. Concrete beam repairs

in stopping ASR; however, it cannot be applied to submerged concrete (FHWA/NHDOT Workshop 2003).

The edges of spalls should be cut to eliminate feathered edges and to ensure the adherence of patching material. Beams and pile caps with extensive spalling on all surfaces may need to be formed and have the mortar mix pressure-injected into the form surrounding the beam. Another common method of restoring the underside of pier and wharf decks with extensive surface spalling uses shotcreting. *Shotcrete*, or *gunite*, is basically a mix of cement, sand, fine aggregate, and water applied by air hose under pressure. Shotcrete also may be modified with polymers or epoxy. It has excellent bonding properties and generally is applied over wire fabric to a depth of 2 in. or more in several passes (ACI 1990, Emmons 1993). The use of synthetic reinforcing fibers in repair concrete, mortars, and shotcrete may significantly improve the adhesion and control cracking for these types of concrete repairs.

Large voids or extensive areas of unsound concrete must be cleaned back to sound concrete and filled with a durable mix of cast-in-place (CIP) concrete, which often requires extensive formwork. Fig. 11-12 shows the formwork set up for an underdeck beam repair. Replacement concrete should be as similar as possible to the cast-in-place concrete in regard to the maximum aggregate size and the water–cement ratio. Fresh concrete is not chloride ionized as the older in-place concrete is and may cause accelerated corrosion of existing reinforcing steel. Coating the exposed steel with a suitable insulation compound, such as an epoxy, can prevent this problem. Contact surfaces may be moistened, but no standing water should be present before placement. Prepacked concrete also may be used for general repairs of larger voids. Its application consists of packing the form with coarse aggregate and then filling the voids with an intrusion grout consisting of Portland cement, sand, water, and a pozzolan and intrusion admixture (Emmons 1993). The dry-pack method consists of ramming a very stiff mortar mix into small openings in thin layers; it is suitable only for small holes, such as form-tie holes and narrow deep slots.

Externally bonded FRP systems for strengthening, repair, and rehabilitation in the marine environment have gained acceptance in large part because of their corrosion resistance and high strength to weight ratio. Examples of their use include supplementing section loss of existing reinforcement or increasing overall strength by bonding FRP strips to the underside of reinforced and prestressed concrete deck planks and wrapping/encasing reinforced concrete beams. FRP repair systems consist of reinforcing fibers, resins/adhesives, primers, putty, and coatings. Common reinforcing fibers include glass, aramid, and carbon. Resins, including primer and putty, generally consist of epoxy, vinyl esters, and polyesters. Protective coatings can vary but are typically polymer or acrylic.

Externally bonded FRPs are available in wet layup, prepreg, precured, and near-surface mounted systems. Wet layup systems include fabrics saturated with a resin in the field and formed around the repaired member. Prepreg systems are fabrics that have been preimpregnated with a resin and generally do not require additional resin application in the field for bonding. Precured systems consist of manufactured composite shapes that can be bonded or fastened to the repaired member's surface. Near-surface mounted systems include round or rectangular bars placed into grooves made in the repaired member and bonded with an adhesive. Design and construction guidance can be found in ACI (2010) and NCHRP (2004). A case study of the application of advanced fiber wrap composites for the seismic retrofit of waterfront structures has been presented by Jiménez and Arnold (2009).

Deck surfaces are subject to surface erosion, generally known as *scaling*, caused by deicing salts or other chemicals, and to wear or damage from traffic and heavy equipment, as well as general weathering. Decks with little and shallow scaling and with only small hairline cracks, or cracks that have been sealed by pressure injection, may be coated with a sealant to prevent moisture from penetrating deeper into the slab. Typical sealants range from linseed oil compositions to epoxy systems, polyurethanes, and other synthetic resins. The use and performance records of the various deck sealant systems are described in detail in Pfeifer and Scali (1981). Where the deck is more surficially deteriorated, an overlay system of CIP concrete or polymer-modified concrete (PMC) may be used. The overlay acts both to seal the underlying structure and to provide a more durable wearing surface. If CIP concrete toppings are used, they usually are on the order of 2.5 to 4 in. thick and must be reinforced with wire fabric or synthetic fibers. Just before placing deck overlays, the original concrete should be saturated with fresh water for 1 to 2 days to ensure that it does not draw mix water out of the fresh concrete and cause shrinkage cracking in the overlay. Overlay cracking can also be reduced by providing a wet surface cure soon after finishing and by controlling the dosage of superplasticizer added to the concrete mix. Concrete crack and shallow spall repairs are addressed in more detail in DOD (2001c).

Much thinner overlays, resulting in much lower dead loads, can be attained by using PMC. Of the many possible PMC mixes, the use of latex-modified concrete is perhaps the most widespread. Suitable latex formulations result in greatly improved bond, tensile, and flexural strengths of cements and mortars. They can be feather-edged to minimum thicknesses of 0.5 in. or less, although the durability of such thin applications under wheel loads is questionable and the higher material cost of the PMC should be considered.

Microsilica (also called silica fume) modified concrete has also been used in deck overlays for waterfront structures. This mix significantly reduces the permeability, increases strength, and increases wear resistance of the new concrete; however, it greatly increases both material and labor costs. If crawler cranes are used on a wharf deck and mechanical wear is a problem, then microsilica mixes may be justified. However, the reduction in overlay permeability may not be of value in overlaying older concrete where chloride penetration and carbonation have reduced the reinforcing passivity. Many concrete contractors are reluctant to use microsilica mixes for deck overlay work because of a lack of bleed water and associated finishing difficulties, which often require the use of wind shelters and fogging sprays, increasing the labor costs on microsilica concrete projects.

Occasionally, attempts are made to patch or overlay concrete decks with asphaltic or bituminous concrete. This is not recommended practice because asphalt can be penetrated by moisture, and moisture and vapor are retained at the concrete surface, thus allowing degradation of the concrete to continue, especially if reinforcing steel is not covered.

As is apparent from the foregoing discussion, a wide variety of polymeric materials and cement admixtures is available for repairs. A detailed discussion of their chemistry and application is beyond the scope of this text. General guidance for the repair of concrete deck systems can be found in ACI (2014a, b) and Emmons (1993) and for the use of epoxy- and polymer-modified concrete in ACI (1993, 2009). A more detailed general treatment of the various concrete repair materials and methods and of the placement of concrete underwater is provided by Thoresen (2014), ACI (2014b, 1998b), and DOD (2001b). It is most important that the design engineer understand the properties, application requirements, and past performance history of a given product. This understanding requires close scrutiny of product manufacturers' literature and documentation. Many of the concrete superstructure repair methods being used for bridge repair in areas where road salt is used

Fig. 11-13. Major rehabilitation of basin dry dock floor and walls involving extensive removal of concrete and granite block facing and installation of prestressed rock anchors to anchor floor and walls against hydrostatic pressures. Note sheet pile cells for temporary end closure

Source: Photo courtesy of Appledore Marine Engineering, LLC

are applicable to superstructure repair for marine structures. All materials must be applied in strict accordance with the manufacturers' recommendations, and surface preparation before application must be clearly specified and carefully inspected.

Major rehabilitation of concrete structures often involves the design of new reinforcing steel and means of properly connecting to the existing structure to remain. Fig. 11-13 shows the rehabilitation of a vintage basin dry dock floor and walls and sill structure at the dock entrance. It involves the removal of several feet of concrete on the existing dock floor down to bedrock and removal of stone masonry block facing and deteriorated concrete on the walls. Posttensioned rock anchors were required to anchor the new reinforced concrete floor and sidewalls against hydrostatic pressures (Elwood and O'Connor 2010). The placement of new concrete of substantial thickness, considered mass concrete, must account for the heat of hydration and possible attendant cracking of the hardened concrete, for which design guidance is provided in ACI (2005).

Steel

Steel members generally are repaired by replacement, plating, sistering, or encasement. In replacing or restoring existing steel members with new plating, careful

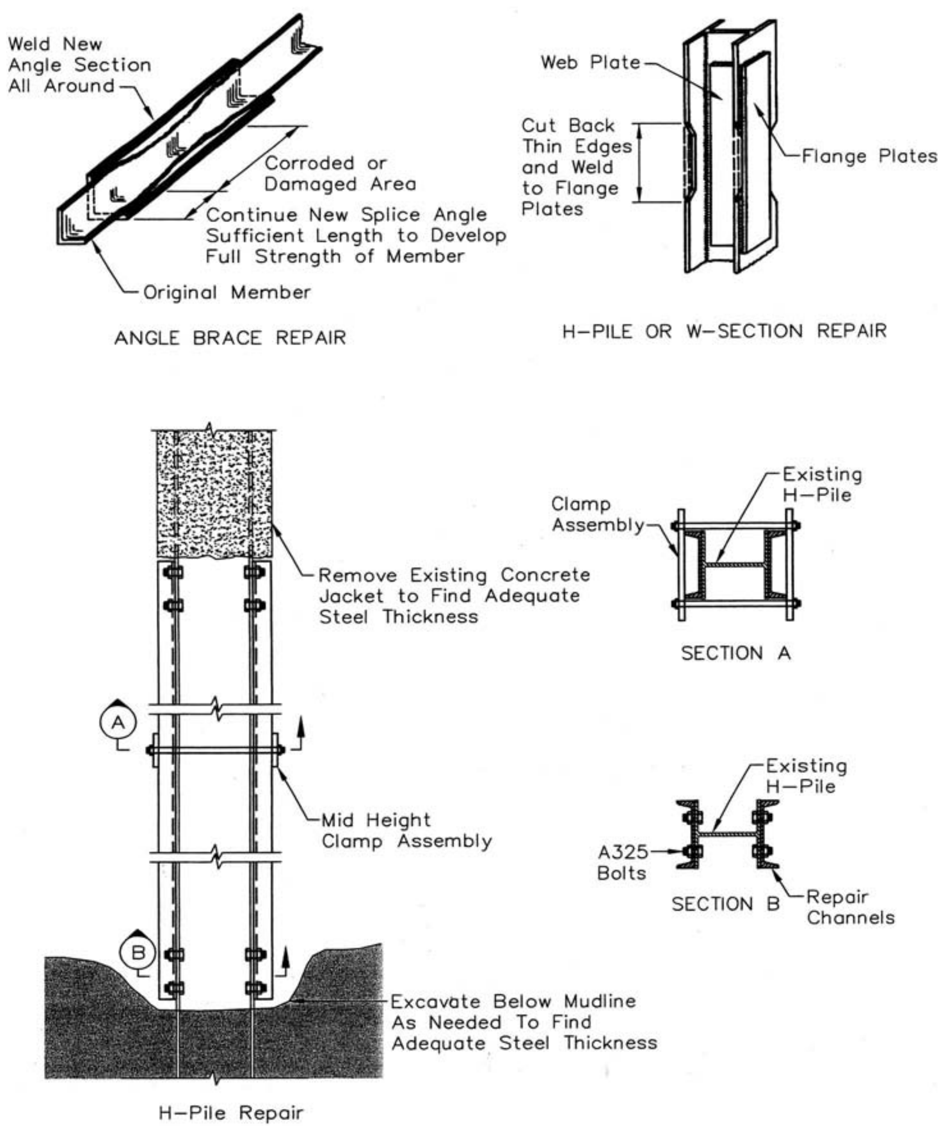

Fig. 11-14. Repair of steel members and piles

consideration must be given to achieving a suitable weld or connection that is both sound and resistant to corrosion. When repairing older waterfront structures, it is often necessary to cut coupon samples from the existing steel members and piles to check chemical and strength properties and assess weldability. New metal, even if of the grade of the existing metal, typically is more anodic than corroded metal, so the new plating may suffer an accelerated rate of galvanic corrosion when submerged. The use of thicker material and fasteners for plating repairs somewhat offsets this effect. Selected methods of steel member repairs are illustrated in Fig. 11-14. To prevent accelerated corrosion of the faying surfaces, the surfaces should be properly cleaned to near-white metal, and continuous watertight welds should be used. Underwater welded repairs and welding under wet conditions are described in AWS (1999) and Fulton (1983). Underwater wet-welding technology has progressed to the point where specialized divers can make structural welds underwater that are comparable to dry welds, using specialized equipment. However, most underwater wet welding by divers is of lower structural quality with lower strength, higher porosity, and less ductility. These common wet welds are adequate for work such as anode attachment and temporary connections. Structural dry-welding can be performed underwater using cofferdams, diving bells, or one-atmosphere chambers; however, this equipment is specialized and often is not readily available for most pier and wharf repair work. Structural steel bolting can be used underwater; however, the standard thin hardened washers and load indicator washers should not be used without cathodic protection. Because of the often irregular fit-up of underwater connections, the structural bolts should be installed to a torque value rather than turn of the nut method, especially if bolting to existing corroded steel, where fit-up may have gaps. Load-indicating washers can be used underwater and can be inspected underwater by divers; however, cathodic protection should be used to prevent washer corrosion. Expansion-type bolts for structural steel connections are available for use where there is access to only one side of the steel member, such as with tube, pipe, and sheet pile, and these bolts are available in stainless steel.

Structural steel members can also be repaired or strengthened by using grout for load transfer past the damaged or deteriorated area. This method may involve filling tubular members with grout or may use an external structural sleeve grouted over the original member (Ricles and Gillum 1996). These techniques are similar to the pile jacketing methods described in Section 11.4. The following list summarizes some practical design considerations to help reduce corrosion problems:

- Select proper materials (e.g., compatible metals and more corrosion-resistant metals where possible).
- Avoid using steel in harsh environments such as tropical climates and exposed bulkheads subject to strong currents and/or wave action with entrained sand and sediments.
- Detail to avoid crevice and local concentrated corrosion (e.g., avoid overlapping plates and make welds continuous).
- Detail for low maintenance (e.g., avoid using back-to-back angles and creating poorly drained areas).
- Design members with extra-thick “corrosion allowance” consistent with anticipated loss rate over the design service life.
- Select appropriate protective coatings and/or corrosion prevention systems and combinations thereof where appropriate.
- Consider effects of corrosion on fatigue life and potential for stress corrosion cracking.

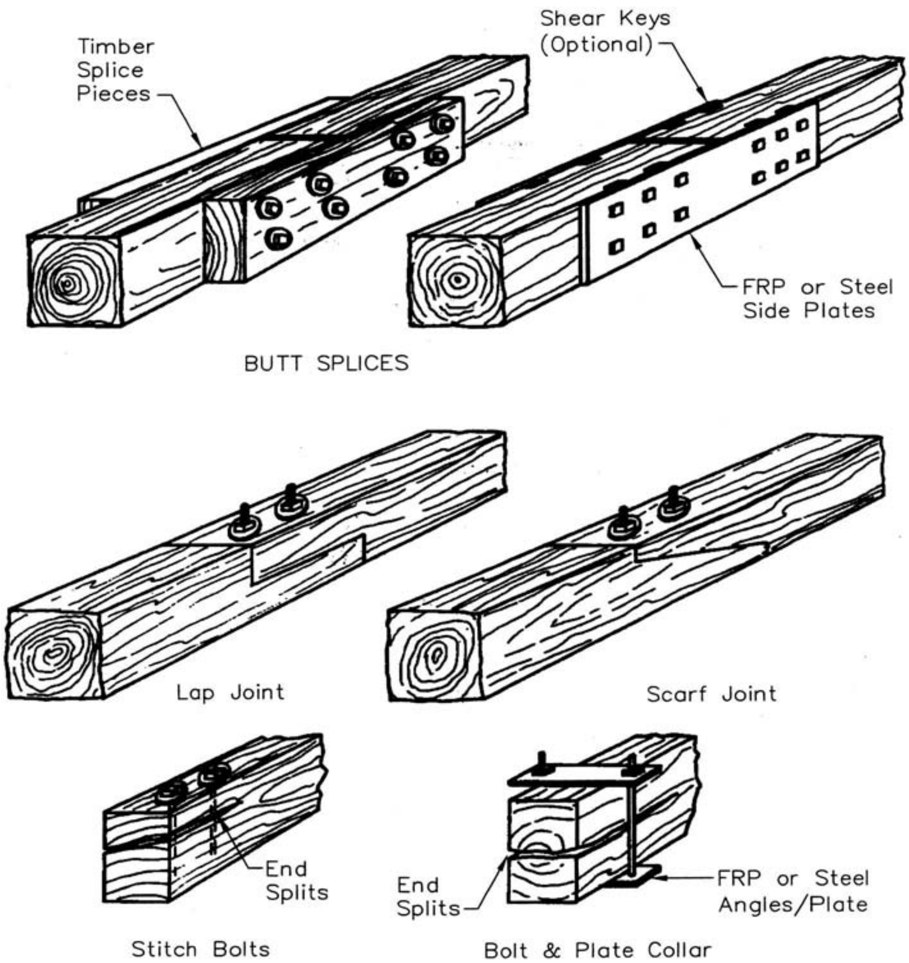

Fig. 11-15. Timber splice and stitch bolt details

Wood

The most common way to repair timber structures is to replace the affected members in kind, but sometimes individual members may be repaired or strengthened by the addition of splice plates or shoring members. Fig. 11-15 illustrates some typical timber splice details that can be used to join replacement sections of affected members. Longitudinal checks or splits sometimes are held together by the use of stitch bolts. Often, it is found that metal fastenings within the tidal and splash zones deteriorate more rapidly than the timber itself. An alternative to these steel connectors is the use of fiber-reinforced plastic (FRP) plate, angles, and channels. They do not corrode and can be drilled in place during installation, thereby eliminating the bolt hole alignment problems common in using multiple steel splice plates or clip angles. The FRP fabrications are now readily available from many manufacturers and are very cost-competitive with galvanized steel fabrications. There are FRP bolts available that may be suitable for shear applications; however, the threads are not strong and they have low tension values. Galvanized bolts perform well in the tidal (upper), splash, and atmospheric zones; however, galvanic corrosion below low water may remove the zinc coating relatively quickly. Since the nuts of hot-dip galvanized bolts are overtapped to allow for the zinc coating thickness, these nuts can become loose when the galvanizing is consumed. Bolted connections of timber in high-corrosion marine connections should use cast iron ogee-type washers rather than flat washers since the thick iron washers are far more corrosion resistant. It should also be noted that the use of stainless steel bolts for fastening wet timber could lead to problems. Stainless steel needs oxygen to maintain passivity; if embedded in an anaerobic condition, such as within wet/submerged timber, then corrosion and pitting occur where they cannot easily be inspected.

Where decay or rot is present, the source of trapped moisture causing the rot should be eliminated if possible before the placing of new timbers. Marine borer attack can be treated by wrapping or encasement if the remaining timber section's strength is adequate. Fire-damaged members should be carefully inspected. If there is sufficient sound material below the surface fire damage, then the member may be allowed to remain as is. In general, all timber used in marine construction should be treated with preservatives, with the exception of some naturally resistant dense tropical hardwoods. Holes and cut ends of treated timber should be field-treated with the recommended brush-on type of preservative. Section 3.5 describes material and treatment requirements for marine construction. The general repair of wood structures is covered in ASCE (1982).

Stonework

Stone masonry work, which typically is made from dense and durable cut granite blocks, seldom exhibits material deterioration problems. Settlements caused by foundation problems and washing out of material through open joints are the most common problems. Open joints may be filled with small stone and mortared. Epoxy or other polymeric grouts may enhance bond and durability. In the sealing of open joints with any kind of mortar, termed *pointing*, extreme caution must be exercised to leave adequate openings or other provision for drainage of water from behind the wall. For repair of larger portions of existing stone walls, concrete is often the most economical repair material. If aesthetics are important, existing stone walls can be rebuilt, being careful to replace unsatisfactory stone with new or recycled stone blocks. Large blocks offer the best wall stability, and contractors prefer using the largest stones available to speed construction and reduce labor costs (Fig. 11-16). New stone is normally supplied in accordance with ASTM C615, *Specification for Granite Dimension Stone*. Granite has a unit weight generally in the range of 156 to 170 lb/ft^3, although "trap rock" may range up to 180 lb/ft^3 or

Fig. 11-16. Granite masonry pier reconstruction

more. The stone blocks in many older structures were doweled or stapled together using iron dowels or cramp irons (large staples). Today, the iron dowels are not commonly available, and steel or FRP rebar may be a better material for dowelling in some instances. Lombillo et al. (2011) discuss the behavior of anchoring reinforcement in stone marine structures based upon mechanical testing results, and Lourenço et al. (2005) and Vasconcelos and Lourenço (2009) present experimental results of the behavior of dry laid stone masonry walls under combined in-plane loading and cyclic loading, respectively, which may be helpful in evaluating the capacity of existing stone walls.

Riprap stone may be displaced by wave or ice action, and when the displaced stone is replaced, consideration should be given to whether the original stone is of adequate weight for its purpose. Occasionally, sedimentary rocks or rubble that are not resistant to weathering may be found in existing riprap slope protection. Such stone or materials should be replaced with more resistant granitic rock, if regionally available. In some locations, hard and durable stone for riprap is not readily available, and precast concrete armor units may be more cost-effective. For in-depth treatment of riprap stone and related stone work, see CIRIA (2007).

Miscellaneous Materials

Other common waterfront materials requiring frequent inspection and replacement include wire rope, chain, connection hardware, elastomeric bearings, and utility piping and systems. Routine inspection and reduction of exposure to deteriorating agents greatly reduces long-term maintenance and replacement costs. PIANC (2009) provides discussion on the use of alternative materials in marine construction. Fender systems and mooring hardware are discussed in subsequent sections.

11.4 Pile and Bulkhead Restoration Methods

In approaching the problem of pile foundation or bulkhead deterioration, one first must determine whether there is sufficient sound material remaining to meet strength requirements or whether structural strengthening also is required. In the first instance, preservation methods such as protective coating and claddings, cathodic protection of steel, plastic or corrosion-resistant metallic wrappings, and encasement in concrete jackets may be applied. Where strengthening is required, general remedies include encasements, posting and shoring, and, in extreme cases, underpinning, which generally requires the driving of additional new piles. For most pier and wharf structures, reconstruction of the entire structure is likely to be more cost-effective than an attempt to underpin the entire foundation. The DOD Unified Facilities Criteria manual UFC 4-150-07 (DOD 2001d) provides guidance for repair procedures for various types of piles in marine structures.

Pile Repairs

Where piles possess sufficient remaining structural strength but must be protected from further deterioration, the use of wrapping, coating, or encasement often is an economical solution. Wrapping materials for pile jacketing of waterfront structures are usually of polyvinyl chloride (PVC) or polyethylene sheeting from 30 to 60 mil (0.03–0.06 in.) thick. The pile surface must be thoroughly cleaned and smooth so as not to tear the wrapping material, which is tightly wound around the pile and secured with corrosion-resistant metallic bands. With proper installation, this method denies free oxygen to the pile surface, thus arresting both corrosion and marine borer activity. There are various proprietary pile jacket systems manufactured for timber piles. The jacket system must provide adequate end and side closure seals to form watertight encapsulation of the pile. If adequate seals are not provided or if holes are torn in the jacket, then water pumps in and out of the jacket with each tide cycle and the internal oxygen levels may be high enough to allow continued marine borer activity (Webber and Yao 2001).

Applying a tight-fitting jacket that incorporates a petrolatum corrosion inhibitor, similar to the tape wrap systems used for corrosion protection of pipelines, can provide corrosion protection of steel piles both above and below water level. After the pile is cleaned, a diver or worker on a float hand-applies a petrolatum primer that displaces water from the steel surface. A petrolatum-saturated fabric is then applied over the primed surface and, finally, an outer protective jacket. This system is well suited to pipe piles, and some manufacturers also make form-fitting jacket systems for H-piles. Unlike concrete pile jackets, these petrolatum jacket systems can be removed for future selective pile inspection. The "soft" nature of the petrolatum type of pile jackets does make them more prone to damage from floating debris; however, they can be applied during cold weather and in cold water, which is not recommended with many other types of coatings. With all of these barrier-type protection systems, the life of the repair is highly dependent on the correct surface preparation and repair installation.

In-place steel and concrete piles can be protected with splash zone coatings above and below water. These coatings are typically applied in 125- to 250-mil (0.125–0.25 in.) thickness by hand tools to displace water off the steel or concrete surface. Offshore structures have been successfully protected by the application of sheet metal, such as Monel and Alloy 400, wrapping. The application of protective coatings, mastic claddings, and cathodic protection systems for piles and bulkheads is discussed in the following sections. Additional description of pile protective methods can be found in DOD (2001a), Whiteneck and Hockney (1989), Dismuke et al. (1981), Watkins (1969), and Johnson (1965).

Posting of piles is a method of pile repair used where severe deterioration is confined to a small area, such as may be the case with timber piles that are rotted at their top butt ends. The decayed area is cut out one pile at a time, and a new piece of timber or steel is wedged into place and secured with splice plates, as illustrated in Fig. 11-17. This method may also be used to replace a portion of pile that has advanced marine borer attack, corrosion, or concrete deterioration. When using steel for the pile post, a telescoping portion of the post can be provided to allow for in-place post length adjustment and the insertion of a jack for preloading the pile. When posting steel and concrete piles, it may be more practical to avoid cutting piles and to apply the post as a structural jacket that is bolted or clamped onto the sound portions of pile, above and below the deteriorated area. To find these portions of the pile with sufficient remaining section, it may be necessary to excavate below the mudline and to chip into the upper concrete-encased portion of pile, as shown in Fig. 11-14.

When deterioration is more extensive, such as severe corrosion of steel piles within the tide zone or marine borer attack of timber in the immersed zone, then encasement in concrete jackets often is used. Concrete jackets may be formed by using steel forms (structural or nonstructural) or leave-in-place fiberglass (FRP), plastic, or fabric forms. Fig. 11-18 schematically illustrates the installation of a concrete jacket. The forms are suspended from the deck above or from falsework and are tightly secured at their base with a baseplate. Concrete then is pumped into the form using the tremie process if it must be placed underwater. Extreme care must be exercised to keep the concrete from falling through the water. Concrete for pile encasements should be of a rich cement mix design, using cement with low C_3A content, such as Portland Type II cement. Aggregates should be well graded and

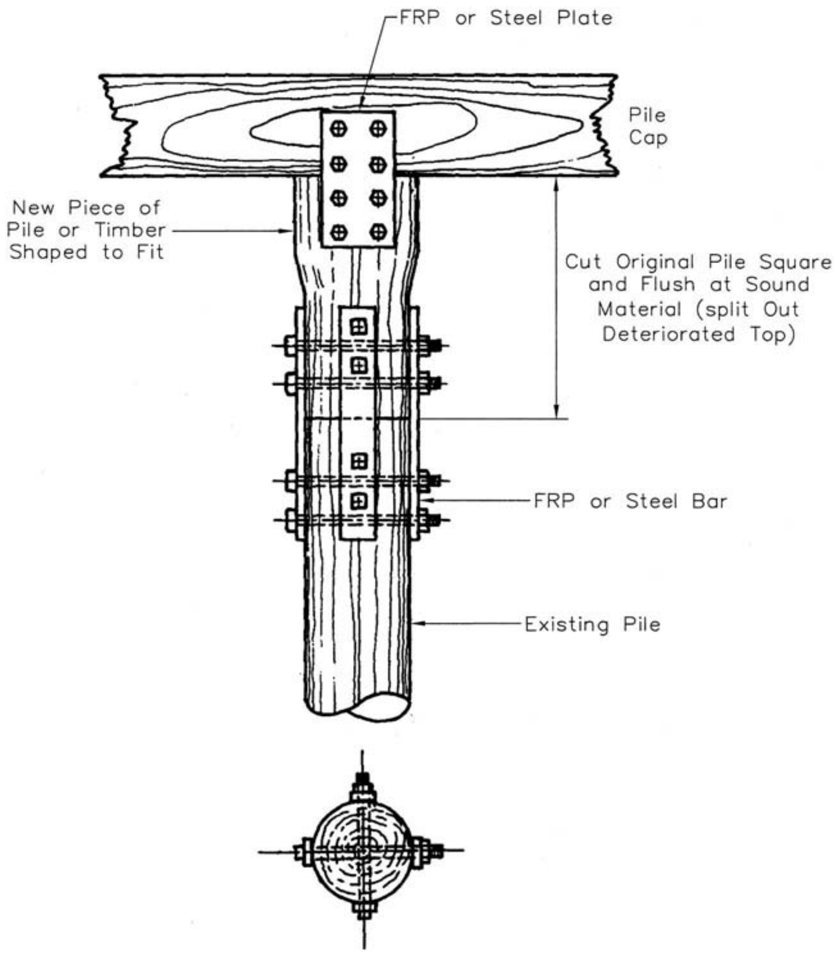

Fig. 11-17. Timber-pile posting detail

durable, mixing water should be free of chlorides, and air entrainment is recommended and is essential in areas subject to freeze–thaw. Pozzolanic admixtures also may be used as fluidizers. Pile shafts, which are applied only within the tide zone, should extend at least 2 to 3 ft below MLW, and the portion of the pile that is exposed at the bottom of the concrete jacket should be coated to avoid creation of a local galvanic corrosion cell (Fig. 11-3). The jacket may be extended to the bottom of the cap or not, depending upon the prevailing conditions and the height of the cap above water. A minimum of 4 in. cover to the steel pile with at least 2.5 in. (3 in. preferred) clear cover to the jacket reinforcing steel normally is required. Welded wire fabric (WWF) is often used as reinforcing steel for convenience in placing and for its bidirectional properties. The washout, impact, and crack resistance of the concrete pumped into these jackets can be improved by adding polypropylene fiber reinforcement to the concrete/grout mix. Antiwashout admixtures are available and

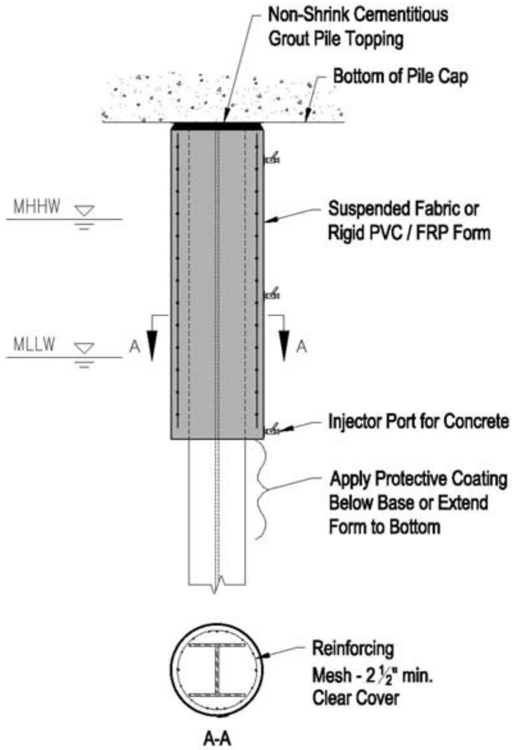

Fig. 11-18. Jacketing of steel pile with concrete

may be cost-effective for relatively low-volume concrete jackets. Encasements for timber piles must extend to at least 1 ft or more below the mudline to prevent *Limnoria* attack. Concrete piles also may be restored by jacketing or by shotcreting.

Where restoration of structural strength is required, it is important that the encasement extend sufficient distance beyond the deteriorated area to transfer the pile's axial, shear, and moment capacity back into the remaining pile section. This method may involve the use of shear lugs to fully develop and/or reduce the bond length required between the repair concrete and existing pile surface. Application of cathodic protection (Section 11.6) to the remaining portions of steel piles below the bottom of the encasement should be considered and can be applied to reinforced and prestressed concrete piles as well (Kessler et al. 2002).

Before using concrete jackets, the engineer should consider the structure's overall rigidity because the concrete jackets may be subject to cracking if large structure deflections and induced moments are possible. The dead weight of concrete jackets should also be considered because the weight of large-diameter concrete jackets can reduce the remaining live-load capacity of piles, especially for low-capacity piles. These concrete jackets are labor intensive and expensive to install, so if many jackets are required, then consideration should also be given to mobilizing pile-driving equipment and replacing the piles through holes cut in the deck, which may be more cost-effective on a lifecycle cost basis. If new timber piles are installed through the deck, the engineer should carefully evaluate the amount of pile head manipulation the contractor is allowed before the pile becomes overstressed by the combined axial and bending loads. Also, if large riprap stone was placed on the slope under a pier or wharf, then the engineer should consider the difficulty of driving the new underpinning piles through the riprap layer. This method may involve opening a path for the pile with a heavy spud or using H-piles or W sections with thick flanges and webs and a compact section.

Another consideration for replacement piles involves selection of the member cross section itself. From a corrosion point of view, for example, pipe piles are superior to H-piles because pipe piles have less surface area, which is also easier to clean and inspect than the H-pile surface. Pipe piles retain coatings better because H-piles have sharp edges where coatings tend to thin out, and corrosion rates tend to be higher near the ends of the flanges. Also H-piles may suffer a more dramatic loss of capacity than pipe piles do because of a reduced radius of gyration about the H-pile weak axis for a comparable average loss rate. This last point is significant in evaluating the remaining capacity of an existing H-pile structure. The radius of gyration for pipe piles is relatively unaffected by corrosion losses.

Bulkhead Repairs

Steel bulkheads also may be restored by concrete encasements, as illustrated in Fig. 11-19. Localized corrosion of sheet-pile webs sometimes is restored by plating. Corroded or damaged interlocks may be restored by welding an inverted angle, or channel section, to the adjoining sheets and filling the void with grout. Additional description of typical steel bulkhead repairs is provided by Kray (1983). When a steel sheet-pile bulkhead has become perforated over large areas, such as along a horizontal band around mean lower low water, for example, it may be impractical to make patchwork repairs or adequate concrete encasement. In this situation, the bulkhead may need to be replaced entirely. This step is often accomplished by driving new sheeting along the outboard face of the existing sheeting. If the existing tiebacks and anchor system are still intact, it may be possible to reincorporate them in the new bulkhead replacement, as shown in Fig. 11-20. Bulkheads may fail because of anchor rod failure and toe kick-out and/or overstressing of sheets or anchors caused by scour or overdredging (Horvath and Dette 1983).

Steel sheet-pile cells typically suffer advanced corrosion near low-tide level and in the splash zone. Structural restoration can be performed by adding steel, concrete, or carbon fiber overlay belting, though driving new steel sheet piling around the existing cell may be more effective on a lifecycle cost basis.

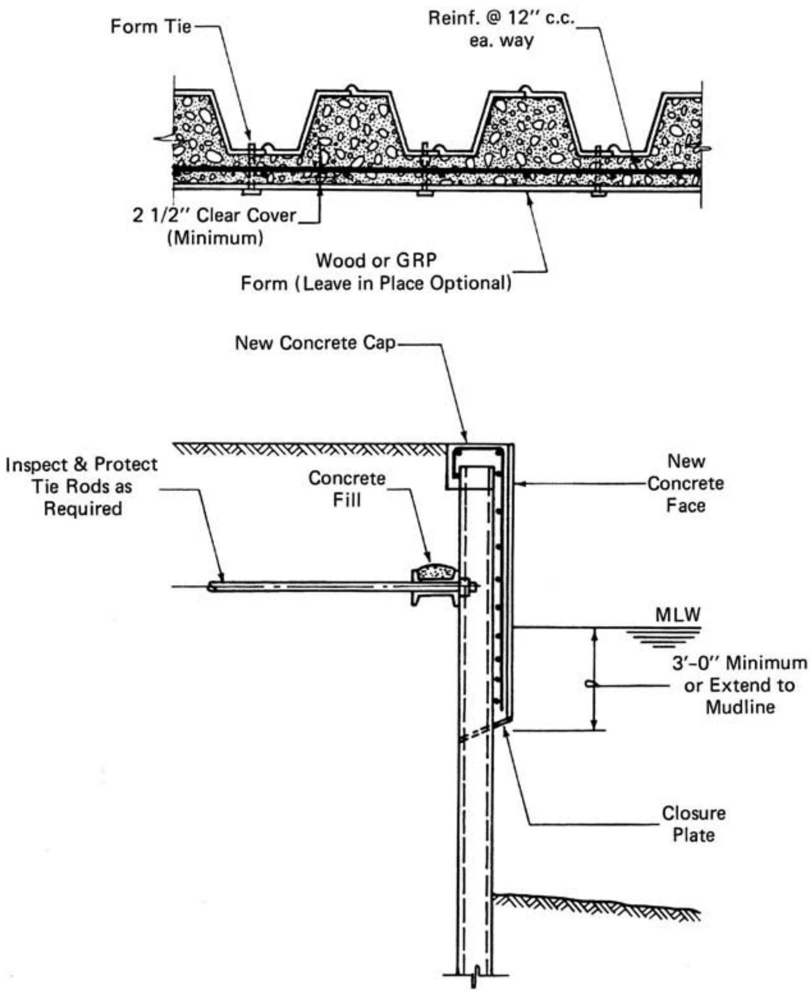

Fig. 11-19. Steel sheet-pile bulkhead repair

11.5 Protective Coatings and Claddings

Protective coatings serve as barriers to isolate the steel substrate from the corrosive action of the environment. As such, a suitable coating system must be impermeable and must possess good adhesion and resistance to the marine environment and to fouling and abrasion. Other considerations include resistance to sunlight and hydrocarbons and compatibility with cathodic protection systems. The ideal requirements for a coating system vary with its class of service (i.e., immersion, tidal, splash zone, and atmospheric). It is essential that any coating system be applied over a properly cleaned and prepared surface. In general, the performance of a protective coating depends upon the surface preparation, application, film thickness, and environmental conditions. The following discussion is intended to review some of

Fig. 11-20. New steel sheet-pile bulkhead driven outside of existing deteriorated bulkhead and using existing tieback anchor rods with new anchor bolts securing new internal wale system to existing wale on opposite sides of sheeting

Source: Photo courtesy of Appledore Marine Engineering, LLC

the more common coating systems specified for marine structures. Selection of coatings for marine structures is covered by Drisko (2008).

Generic coating systems are specified by the Society for Protective Coatings, formerly known as the Steel Structures Painting Council (SSPC), in their two-volume painting manual (SSPC 2008), which also covers good painting practice. The SSPC manual includes many standards for surface preparation, such as sandblasting and high and ultrahigh-pressure water jetting. Water jetting is especially well suited to marine structure repair since blast grit does not have to be collected, but water jetting does not create a roughened steel surface, and the existing steel surface profile may not be adequate for some coatings. Recommended practice for the corrosion protection of marine structures is provided by the National Association of Corrosion Engineers (NACE 2003, 2002, 1994).

Coal tar epoxies are among the most common systems used for immersion and splash zone service. They consist of a two-component, catalyzed thermosetting resin, usually either amine- or polyamide-cured. The epoxy has good adhesion characteristics, is tough and resistant to chemicals and solvents, and combined with coal tar has excellent water resistance. Coal-tar epoxy coating systems should be applied over a near-white-metal sandblasted surface in accordance with SSPC (2008), to a minimum dry-film thickness (dft) of approximately 16 mil. Up to 20- to 24-mil dft may be specified for splash zone and severe exposures. The lifetime of an epoxy coal-tar system is directly proportional to the film thickness, and a properly applied coating should give 10 to 15 years of service. Straight coal-tar coatings are inexpensive and water-resistant but lack the toughness, durability, and solvent resistance of epoxy- or polyurethane-modified systems. Other drawbacks of coal-tar systems are their easy penetration by fouling organisms and their poor performance when used in conjunction with cathodic protection systems. The future recoating of members originally coated with coal-tar epoxies is somewhat compromised by the poor adhesion of most coatings onto a coal-tar epoxy substrate. Straight epoxy systems give good service, and some *polyamide-cured epoxies* have been developed that cure under moist conditions and even underwater. These coatings, which generally are more expensive than coal-tar epoxies, also must be applied over a near-white-metal sandblasted surface.

Fusion-bonded epoxy coatings are one-part, heat-cured systems, as used in epoxy-coated reinforcing bars. The process also can be applied to pipe and structural shapes, and the finished product offers durability and uniformity of coating. As the process involves heating the member to high temperatures and applying a powder under controlled conditions, it is not amenable to field application. Fusion-bonded epoxy coatings generally do not have good ultraviolet light resistance, and they may need to be topcoated if UV deterioration is a concern.

Polyethylene coatings applied by a heat process constitute a relatively recent and promising coating system. Polyethylene coatings applied in varying thicknesses have been used on offshore pipelines and pipe-pile structures, and long lifetimes, of 40 to 50 years, are predicted. Pipe-pile ends that are field-spliced during driving are fitted with field-applied, shrink-fit polyethylene collars. This coating system can be used for replacement pipe members; however, it is not suitable for application to in-place members.

Other coatings suitable for immersion service include phenolic mastics, chlorinated rubber compounds (often used inside ballast tanks), and topcoated metallized zinc or aluminum systems. *Metallizing* is a process of spraying molten metal particles (usually zinc, aluminum, or zinc/aluminum alloy) onto a prepared surface. Aluminum metallizing is recommended for immersion service. Metallized surfaces must be topcoated, usually with a vinyl, epoxy, saran, or furan topcoat, to keep the zinc surface film from eroding rapidly.

Zinc-rich coatings often are used as prime coats for epoxy, vinyl, or acrylic coatings and also may be used by themselves in atmospheric exposures. They offer good splash zone and freshwater protection as well. Inorganic zinc and organic zinc coatings are so named by the nature of the material used as a binder. They are usually applied in relatively thin coats of 2- to 5-mil dft. Generally, 1-mil dft should be allowed for each year of life required. Zinc-rich paints provide cathodic protection to the metal substrate when the paint film is scratched. Organic zincs have better film-forming properties than inorganic zincs and are generally more suitable for primers. Typically, the organic zinc primers are not used for immersion service.

Galvanizing is a process of coating iron and steel with zinc, typically by the hot-dipped galvanizing (HDG) process after fabrication by dipping the items to be coated into a vat of molten zinc metal. The zinc provides a two-part protection, both as a barrier coating and also as a sacrificial anode if the coating is scraped or damaged because it is anodic to iron and steel. The galvanized coating consists of a series of zinc/iron alloy layers that are corrosion inhibitive with an almost pure zinc top layer. HDG coatings are specified under ASTM specifications A123/A123M for iron and steel products and A153/A153M for iron and steel hardware. Detailed information on HDG can found in the many publications of the American Galvanizing Association (AGA) (see Appendix 3 for web address), and an introduction to specifying HDG can be found in AGA (2012). Galvanizing should in general last many years in most marine atmospheric and splash zone environments; however, it is not acceptable for continuous immersion service because the zinc soon goes into solution to form zinc chloride. The HDG process has little or no effect on the strength of most mild steel products; however, high-strength steels with ultimate tensile strengths of 150 kip/in.^{2}, certain high-strength bolts such as ASTM A490, and certain wire rope and chain products may suffer from hydrogen embrittlement due to chemistry changes associated with HDG cleaning preparation process and therefore should not be galvanized.

Vinyl coatings have excellent resistance to high humidity and salt atmospheres and sometimes are used for immersion service as well. *Polyurethanes* are known for their toughness and abrasion and impact resistance, and they are very flexible, but their adhesion properties are only fair, and aromatic urethanes, in particular, have poor resistance to weathering.

Polyurea coatings appear to be very promising for coating new steel and in-place dry or damp steel above low water. This coating has good adhesion, abrasion resistance, quick cure, and flexibility. The drawback of polyureas is the need for specialized application equipment and an installed cost many times higher than coal-tar epoxy.

Claddings are extra-thick coatings designed to be field-applied by spraying, troweling, or a gloved hand, usually for splash zone protection. They can cure under moist conditions, and some splash zone mastics can be applied underwater by a diver's hands. The underwater curing materials typically are two-component, 100% solids, nonshrinking polyamide epoxy compounds. They normally are applied to 125- to 250-mil dft, with underwater applications often limited to patching or narrow zones. They should be applied over a near-white sandblasted surface and have fair to good impact and abrasion resistance. These coatings also will bond to concrete, wood, or fiberglass surfaces.

Problems with coating systems include skips, "holidays" (local areas of thin or missing coating), blistering, and debonding, all of which are most often associated with application and usually require consultation with the coating manufacturer. Damage to coatings can also occur during installation or by abrasion or by a poorly adjusted impressed current cathodic protection system.

There are many other coating systems with specific advantages and disadvantages for a given application and a myriad of trade name products for the engineer to consider. It is important to review the product literature of different manufacturers and, finally, to consult the proposed manufacturer's technical staff prior to specifying a coating system for a specific project. A comprehensive treatment of protective coatings for marine structures can be found in Drisko (2008), Whiteneck and Hockney (1989), and Dismuke et al. (1981). When evaluating coatings, keep in mind that it may be less expensive to use a thicker, uncoated steel member with a corrosion allowance than it would be to apply an elaborate coating system.

11.6 Cathodic Protection Systems

Cathodic protection (CP) systems provide corrosion protection to immersed steel surfaces by imposing a reverse electric current, in essence making the metal act as a cathode rather than an anode (refer to Section 11.2). Because seawater acts as the electrolyte, the system works only when the metal surface is submerged. However, partial protection within the tide zone is provided during high water. Cathodic protection systems are of two basic types: the galvanic, sacrificial anode (SA) system and the impressed current (IC) system. Fig. 11-21 schematically illustrates cathodic protection system installations.

In the galvanic anode system, sacrificial anodes of aluminum, magnesium, or zinc are attached directly to the piles, thus inducing a flow of electrons away from the anodic material and toward the cathodic pile surface. Anodes must be properly sized and located to ensure a uniform distribution of current density over a planned structure life or anode replacement time.

In an impressed current system, a DC electric current is applied to all the metal through electric bonding cables. Anodes of graphite, silver/lead, or platinized metals are either suspended from the deck, pile-mounted, or bottom-supported in wells. As a rule of thumb, an electric current of 1 ampere (amp) flowing for 1 year corrodes away 20 lb of steel. Usual current densities range from 0.005 to 0.06 amp/ft^2 of protected surface. The electric current raises the pH at the metal surface, causing precipitation of calcareous (chalk) deposits. High current densities result in a hardened chalk, and low densities result in a soft chalk that is easily washed away. It is essential that all steel be bonded to the circuit. This step must be done during the original construction, and it is recommended that any new piers with concrete decks on steel piles be completely bonded even if there is no intention to install rectifiers and activate an impressed current system initially. Compatibility of cathodic protection systems, especially impressed current systems, is an important consideration. Deepwater structures may be protected solely by cathodic protection below low water and with splash zone coatings from the tide zone to the deck.

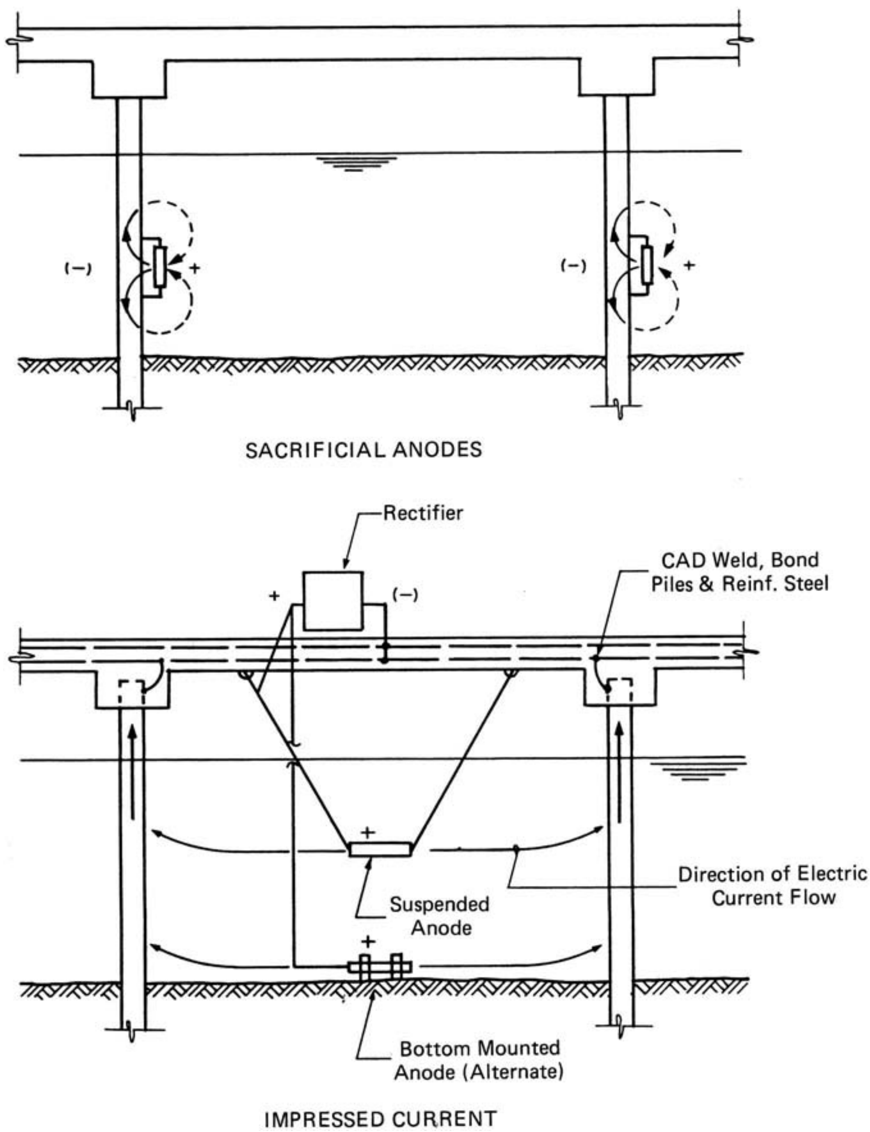

Fig. 11-21. Cathodic protection systems schematic

Important factors affecting the design of cathodic protection systems include the resistivity of the water as measured by its salinity or dissolved solids content, water temperature range, dissolved oxygen content, pH, tide range and currents, pollutants, and bottom soil characteristics. The resistivity of most continental U.S. coastal waters is generally in the range of 20 to 30 ohm-cm, requiring mean applied current densities in the range of approximately 5 to 10 mA/ft^2, depending upon other local environmental factors. At Cook Inlet, Alaska, the resistivity is much higher, around

50 ohm-cm, requiring mean applied current densities of around 35 mA/ft^2. Initial current densities, generally higher than final current densities, are required to hold the structure at protective potential until full polarization has been accomplished and calcareous coatings have been built up by the design current density. Representative CP design requirements for selected offshore and harbor locations are provided by Baboian (2002). The required current density may be further affected by turbulence such as that caused by structure vibrations, prop wash, storms, and so on, which increases the oxygen content at the pile surface and thus the required current density. Aged structures with corroded, pitted metal surfaces are more difficult to protect than new structures and generally require higher current densities. Electrical interference from stray currents also can have an important effect on system design and performance.

Cathodic protection systems are generally designed by specialists with experience in marine structures; however, all port engineers should be familiar with their operating principles and application. DOD (2005, 2003) and NACE (2002) provide guidance for the design and operation of cathodic protection in general, and NACE (2003, 1994) and Lehmann (1979) provide guidance specific to marine structures. Wagner and Fitzgerald (1979) present an instructive case history of retrofitting a deepwater wharf with an impressed current system.

11.7 Fender Systems

Fender systems are subject to mechanical damage caused by accidental vessel impact and the cumulative effects of frequent routine berthing (Padron and Han 1983). Timber fender piles treated with high retention levels (2.5 lb/ft^3) of chromated copper arsenate (CCA) preservative, as specified for bearing piles in marine environments, become somewhat brittle, making them more susceptible to impact damage. Depending on the level of marine borer activity and frequency of fender damage, the timber fender piles could be replaced with timber treated to lower levels of preservative retention, such as 1 lb/ft^3, or even by using untreated timber.

Recycled polyethylene plastic “lumber” and piles have been used for replacement of timber members in some fender systems. Unreinforced plastic members and piles are generally unsuitable for fender system applications. Some plastic lumber is reinforced with fibers and has improved stiffness and bending properties. Plastic lumber and piles that are reinforced with steel or FRP rebar have structural properties more similar to timber. Depending on the stress level and stiffness needed, a fender system originally built with timber can be replaced with composite members and piles. Presently, the initial cost for reinforced composite lumber and piles is higher than for timber. The economic justification for using plastic lumber and piles must be made on the case of a long service life (Alling 1996).

Rubber fender units suffer from gradual wear, overload damage, UV exposure, and ozone attack. Preformed rubber units may crack along their juncture with metal baseplates to which they are bonded and often need to be replaced when such cracks or baseplate delamination occurs. Fenders that have been subject to frequent overloads should be replaced with more resilient, higher capacity units. Rubber stiffness properties can be varied over a relatively large range to suit particular applications. The fender manufacturer's technical staff should be consulted before specifying replacement units.

In the rehabilitation of fender systems, the present uses should be evaluated to verify that the original fender system design is adequate for the vessel presently using the facility. Over the life of a pier or wharf, it is not uncommon for the ship size to increase. The use of modern high-energy-absorption resilient fender units can greatly reduce vessel-berthing reaction forces; in fact, fender systems often can be designed around the lateral load capacity of the existing pier and still accommodate larger ships, as described in Section 5.6. The general maintenance of fender systems and camels is covered by NAVFAC (1990).

11.8 Mooring Hardware

Deterioration of mooring hardware is a common sight at marine facilities. Often, in-kind repairs are performed that do not solve the problems associated with poor detailing in the original design, and the repairs may fail within a few years. A classic example of this is the continual cracking and spalling of concrete slabs and the sides

Fig. 11-22. Typical concrete pedestal damaged by corrosion expansion of embedded cast-steel bollard

Fig. 11-23. Failed bollard casting caused by initial defect in base that corroded over time, initiating cracking and separation under high mooring line load

of concrete pedestals that occurs when cast-steel mooring fittings are embedded into the concrete without adequate corrosion protection or sealing, as shown in Fig. 11-22. Water migrates into the cold joint between the painted mooring fitting and the concrete. Over time, the cast-steel mooring fitting begins to corrode, and the corrosion scale expands in volume and crack or spalls the concrete.

There are solutions to this problem, including hot-dip galvanizing the mooring fitting when new or mounting the mooring fitting on a grout bed, such that the fitting is not embedded in the concrete. Note that most cleats, bitts, and bollards are cast hollow and filled with mortar or concrete when installed, which precludes the option of hot-dip galvanizing when reusing existing castings; however, existing mooring hardware can be metallized to provide corrosion protection.

There have been many instances of hardware castings themselves failing, often an older cast iron fitting that likely had an initial defect that was subject to corrosion and/or repeated high stress over time and was finally loaded in excess of its reduced capacity (Fig. 11-23). These are typically brittle failures that occur suddenly and unpredictably. Castings may also be subject to local wear such as by wire rope mooring lines without chafe protection, which eventually cuts a readily observable groove into the metal.

Another common form of deterioration of mooring hardware is the failure of the anchor-bolt nut pocket sealant, which allows the ingress of water and corrosion of the anchor nuts and bolts. Traditional nut-pocket sealants have included molten lead and molten tar. The molten lead sealant contracts as it cools and can pull away from the sides of the nut pocket, and thus needs peening with a hammer to reexpand the lead. Often, this labor-intensive peening process is omitted during

Fig. 11-24. Testing of postinstalled anchor bolts using a hydraulic jack

Source: Photo courtesy of Appledore Marine Engineering, LLC

construction, and the lead forms a poor sealant. The tar sealant typically hardens and shrinks with age, eventually cracking and failing. Solutions to these nut-pocket-sealant and anchor-bolt-corrosion problems include the use of more modern sealants, such as urethane sealant, and the use of hot-dip galvanized or stainless steel anchor bolts.

If there are concerns about the condition and capacity of a mooring fitting and its anchorage, then the fitting can be load tested. A method for pull testing individual anchor bolts is presented in UFC 4-150-08 (DOD 2001c); however, this method does not test the fitting or its foundation. When new replacement hardware is called for, it is typically anchored with postinstalled anchor bolts that should be tested before securing the new hardware, as shown in Fig. 11-24. UFC 4-150-08 (DOD 2001c) describes the use of cranes and winches to perform load testing of mooring fittings; however, this method can be extremely hazardous if a rope or the mooring fitting fails, releasing the stored elastic energy. Also, the pulling capability of most readily available winches, cranes, and tugs is inadequate to test mooring fittings over 40- to 50-ton capacity. A general rule of thumb for tug bollard pull is about 10 to 15 tons of bollard pull per 1,000 hp. See Section 6.11 for further discussion of tugboats and bollard pull capacities.

Because of the risks associated with pull testing mooring fittings, it is generally recommended that load testing mooring fittings and associated foundations be done by jacking a horizontal load (usually directed offshore) into the fitting using a hydraulic jack, jacking strut, and a reaction mass, such as heavy construction equipment or concrete blocks. The loads and movement of the fitting can be monitored using remote readout gauges, and the load can be rapidly reduced when excessive movement or failure is observed. McGeady (2013) describes the development and use of a portable steel frame for in situ testing of bollards up to around 100 tons. There are commercially available bollard load testing services available as well (see Appendix 3).

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CHAPTER 12

Facility Inspection and Assessment

Appledore Marine Engineering, LLC

12.1 Purpose and Scope of Inspections

Waterfront inspections play a critical role in helping a waterfront structure attain its intended design life. Periodic above-water and underwater inspections help assess the condition of a structure and prioritize repairs, avoiding unanticipated interruptions to operations and avoidable repairs.

Waterfront structures are exposed to an aggressive environment, often resulting in accelerated deterioration. Above-deck deterioration is frequently observed by workers and maintenance personnel; however, deterioration often goes unnoticed under-deck and below water. A routine waterfront structural inspection helps identify deterioration early on to allow the facility time to budget and plan maintenance and repair projects or validate existing maintenance programs that may be in place. Routine inspections serve as a baseline for future inspections to help project and analyze rates and patterns of deterioration and allow for economical and efficient repairs.

A valuable tool in the inspection process is the development of a maintenance and operational manual. A maintenance and operational manual establishes best practices; it identifies key elements to be inspected, including the loading criteria and design life of the structure, the operational requirements, and suggested maintenance practices. It is important that that the engineers performing the routine inspections be familiar with these elements so that they may determine if any corrective measures are required for the facility. In general, the team of design engineers is not the same team performing the routine inspections; therefore, the manual is also a working reference for the inspection team regarding the design objectives of the structure.

There are three basic types of waterfront inspections: routine, design repair, and special. Routine inspections generally account for the majority of waterfront inspections performed. In general, a routine inspection is a snapshot in time; its intent is to give an overall condition and provide operational ratings for structures at the facility. The scope of special and design repair inspections varies, depending on the site and intent of the inspection. Table 12-1 illustrates the different types of inspections, their general objectives, and a guideline for when they should be completed.

Routine inspections are categorized into three levels of effort to better help the engineers use their resources and focus their efforts. Underwater inspections are significantly more difficult than above-water inspections because of low visibility, current, restricted access, and marine growth. In certain scenarios, a remotely operated underwater vehicle (ROV) may be used in place of divers. This is particularly useful in deep or penetration dives.

The scope of a routine-level inspection typically includes a 100% Level I inspection, 10% Level II inspection (Fig. 12-1), and 5% Level III inspection. Table 12-2 provides a description of the three levels.

Level III inspections are performed over a small sample set of elements. The findings from these inspections are extrapolated over the entire structure to establish reasonable quantities for repair estimates and load restrictions.

Table 12-1. Types of Waterfront Facility Inspections

Inspection Type	Inspection Objective	Inspection Frequency
Routine	A general inspection of each structure to develop overall condition. Results of the inspection are used to formulate repair and maintenance recommendations and any requirements for follow-up inspections (design repair or special). The inspection is not intended for use in design of repairs.	Performed regularly throughout lifetime of structure. Refer to Table 12-3 for typical inspection intervals.
Design repair	Collect detailed information for defects relevant to repair project, which is used to prepare bid documents for a repair project at the facility. The inspection determines the most effective repair method.	Performed before repair or rehabilitation projects.
Special	Collect detailed information regarding a specific event or issue that was noted. Some types of special inspections include: postevent (because of storm or impact damage) and modifications in facility use.	Performed when detailed information is needed to analyze the structure, or after a particular event that causes a need for a structural evaluation.

Fig. 12-1. Engineer-diver completing a Level II and Level III inspection of a steel pile

Source: Photo courtesy of Appledore Marine Engineering, LLC

Table 12-2. Levels of Inspections

Inspection Level	Description
Level I	A visual inspection or a tactile inspection that looks for major deterioration or overstressing.
Level II	Inspection is focused on finding surface deterioration that would otherwise be hidden by marine growth or corrosion scale. Marine growth is typically removed in 1-ft bands at the mudline, midwater, low water, and splash zone. Locations can be modified based on experience with the type of structure or environment. Areas of increased stress, such as welds and bolted connections, may be selected for cleaning.
Level III	A detailed inspection involving nondestructive or partially destructive testing. The intent of this inspection is to look for hidden or interior deterioration to the elements. It can also be used to collect in-depth defect information that can be extrapolated throughout the structure.

The frequency of routine inspections varies based on material, condition, and environment. Table 12-3 is from ASCE (2001) and identifies the time interval recommended between routine inspections. General guidance for performing waterfront facilities inspections and assessments can be found in ASCE/COPRI (2015) and for the inspection of dry dock facilities, ASCE/COPRI (2010).

Table 12-3. Frequency of Inspections

Condition Rating from Previous Inspection		Construction Material: Unwrapped Timber or Unprotected Steel (No Coating or Cathodic Protection)[4]		Construction Material: Concrete, Masonry, Wrapped Wood, Protected Steel, or Composite Materials[4]		Channel Bottom or Mudline Scour[5],[6] (Soundings[7]/Direct Observation)	
		Benign[2] Environment	Aggressive[3] Environment	Benign[2] Environment	Aggressive[3] Environment	Benign[2] Environment	Aggressive[3] Environment
6	Good	6	4	6	5	6/6	2/5
5	Satisfactory	6	4	6	5	6/6	2/5
4	Fair	5	3	5	4	6/6	2/5
3	Poor	4	3	5	4	6/6	2/5
2	Serious	2	1	2	2	2/2	2/2
1	Critical	0.5	0.5	0.5	0.5	1/1	0.5/1

1. The maximum interval between routine inspections may be reduced based on extent of deterioration, anticipated deterioration, and importance of the structure. Intervals may be increased for atypical cases where special construction materials are used. Regulations may dictate a maximum inspection interval.
2. Benign environments include freshwater with low to moderate currents (currents < 0.75 knots)
3. Aggressive environments include brackish water, seawater, polluted water, or waters with currents exceeding 0.75 knots. Facilities that handle chemicals containing elements detrimental to the structure's durability, such as chlorides, sulfates, or alkalis, are aggressive environments.
4. The intervals indicate requirements for sounding timbers.
5. The intervals indicate requirements for direct observations of the bottom for scour.
6. Two maximum intervals are shown, one for the assessment of construction material (wood, concrete, steel, etc.) and one for scour (last two columns). The shorter interval should be used. Sounding may be performed at the time of the above water inspection.
Source: ASCE (2001).

The inspection of waterfront structures requires a specialized inspection team that has adequate knowledge and experience in the waterfront environment. Inspection team personnel should meet minimum qualifications to perform inspection tasks; they should be able to evaluate deterioration, and formulate condition assessments and repair recommendations. All diving must be completed in accordance with the federal government's safety standard, OSHA 29 CFR Part 1910, Subpart T, and it is often also required to follow the U.S. Army Corp of Engineer's safety standard, EM385-1-1, Section 30 (2014).

The inspection team should be led by and under the direct on-site supervision of a team leader (TL), who reports to the project manager. The TL is on-site for the duration of the inspection and is responsible for directing the team to help ensure that all required tasks are completed. The TL should complete a significant portion of both the underwater and above-water inspection. The TL is responsible for communicating with the facility owner/operator to discuss the inspection procedures in terms of what the inspections involve and what deliverables to expect. According to the MOP 130 (ASCE 2015), the TL should be a registered professional engineer (P.E.) with a minimum of 5 years' experience in similar work and a certified commercial diver, and all members of the underwater inspection team should be certified as commercial divers and trained in structural inspections. Whereas commercial diver certification is certainly beneficial and should be required at deepwater and offshore sites and under particularly hazardous conditions, such as penetration dives, underwater structural inspections can and have been safely performed by experienced and qualified P.E. divers at inshore, protected harbor locations. MOP 101 (ASCE 2001) does not include the commercial diver certification requirement but does emphasize the importance of diver training and experience. Additional discussion of inspection team qualifications can be found in the MOP 130 (ASCE 2015).

Communication with the owner is critical throughout the inspection process. While developing the scope, it is important that the owner understand the type of inspection to be performed along with any limitations of that inspection. The structures to be inspected and quantity of elements should be clearly defined, and all deliverables should be outlined within a well-defined scope of work. When the inspection team arrives on-site, it is important that a preinspection meeting, or "in-brief" be scheduled between the facility owner/operator and the inspection team. The in-brief is an opportunity to discuss any structural issues that may be ongoing at the facility, historical repair projects, and coordination of daily work at the facility, including safety procedures and protocols. At the completion of the inspection and before the inspection team leaves the site, a postinspection meeting, or "exit-brief" should be conducted by the TL. During the exit-brief, the team presents a general summary of the findings and confirms that all of the owners' concerns were addressed. The exit-brief is an opportune time to discuss with the owner/operator issues and/or recommended loading restrictions.

Because of the nature of work performed while conducting a field inspection, an inspection team is highly recommended (and in some cases required by law) to satisfy the necessary insurance coverage. Refer to MOP 130 (ASCE/COPRI 2015) for more information regarding insurance requirements.

12.2 Inspection Methods and Procedures

Inspection teams require competent engineers who understand the methods and procedures for inspecting various materials encoutered within waterfront structures. Numerous methods can be used to inspect each element of a structure. This section discusses some common findings; it also addresses industrywide practices used when inspecting waterfront structures.

A waterfront facility site inspection is typically separated into four categories: site, facility, structure, and element. Fig. 12-2 provides a typical breakdown of a waterfront facility site inspection.

When preparing for an inspection, the TL should consider all avenues of approach to the structure. Without proper planning, access to certain areas of a structure can be a challenge, and this challenge may prevent a successful inspection. For example, access to elevated decks and/or the underside of the decks is critical for a tactile inspection. To access elevated surfaces, an inspector may require floats, boats, or temporary scaffolding, or he or she may have to coordinate inspections around high and low tides to thoroughly evaluate the elements.

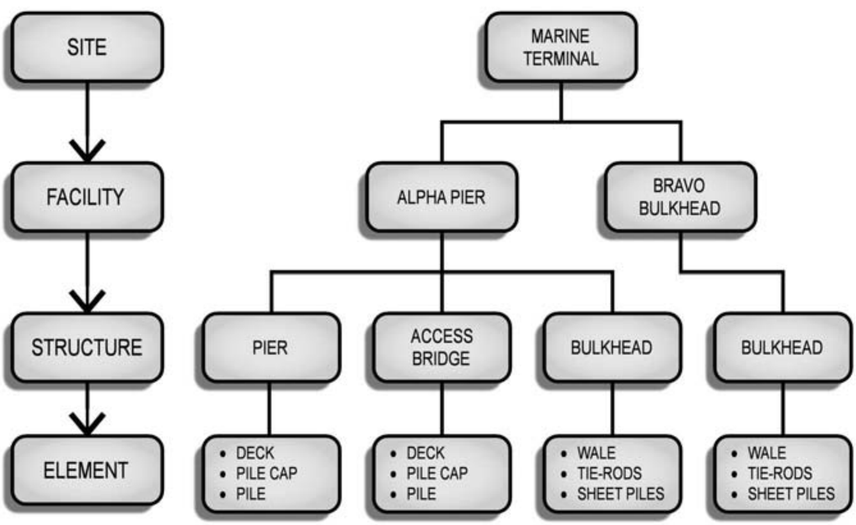

Fig. 12-2. Waterfront facility inspection organizational breakdown

Source: Courtesy of Appledore Marine Engineering, LLC

Before completing an inspection, it is important to review available archive data on the structure, including design documents, repair documents, and prior inspection reports. Reviewing archive design and repair documents allows the inspection team to identify structure size, material types, layout, and loading criteria, and prior inspection reports, if available, provide insight into prior defects and recommended maintenance.

Once archive documents have been reviewed, the inspection team can appropriately prepare for the on-site inspection and assemble the tools necessary to complete the job; these tools tend to vary by site. Common tools routinely used include hammers, measuring devices, material testing equipment, cameras, and other recording instruments.

A routine level inspection should evaluate all major element groups, including but not limited to above-deck superstructure, under-deck superstructure, above-water and underwater substructure, mooring hardware, fender systems, and utilities. Each element of the structure should be inspected following Level I, II, and III criteria. The data for each element should be reported using a standardized reporting method, which is discussed later in this chapter.

Waterfront structures are constructed from a variety of materials; the most common are concrete, steel, and timber. When inspecting concrete elements, typical defects consist of cracked, spalled, or delaminated concrete and broken elements. A Level I inspection identifies gross defects, including spalls, exposed reinforcing, cracks, and visible broken elements. A Level II inspection identifies defects such as cracks or material loss covered up by marine growth. Level III testing methods commonly used for concrete include chain dragging, sprocket wheel rolling, measuring crack widths, rebar locating, and collecting of concrete core samples. When using a chain drag or sprocket wheel, a portion of the deck is selected and the area is sounded by dragging the inspection tool over the concrete selected for inspection. Delaminated or spalled concrete exhibits a hollow or dull sound, whereas areas of sound concrete have a solid sound. The surrounding area must be relatively quiet and free of standing water to perform chain dragging or sprocket wheel procedures. A checklist and means of identifying the various forms of concrete deterioration can be found in ACI (2008).

Within selected areas for Level II and III inspection, the cracks should be mapped for size and general location. The American Concrete Institute (ACI) identifies crack widths of up to 0.006 in. as acceptable for reinforced concrete under service load exposed to seawater and seawater spray, wetting, and drying (224R-01: *Control of Cracking in Concrete Structures*, ACI 2001). Though a crack can be considered acceptable, it should be mapped to assess any changes in condition. Cracks greater than 0.006 in. may allow for ingress of chloride ions. Reinforcing steel encased in the concrete reacts with the ions, which may cause the concrete to delaminate or spall. Additional testing that may be completed as part of a Level III inspection include rebar location, if rebar spacing is unknown, or taking concrete cores to examine chloride ion penetration and perform petrographic analysis.

Asphalt wear courses are commonly found on top of timber or concrete decks. The installation of asphalt over the structural deck elements can mask defects and make evaluation difficult.

Typical defects found during inspection of steel elements consist of corroded, bent, or buckled members and bolts. During a Level I inspection, gross defects, such as buckled members and severe corrosion, are identified. A Level II inspection reveals pitting, corrosion holes, and section loss (Fig. 12-3).

The removal of corrosion by-product may be accomplished with a hammer. If the by-product is "cemented on," larger mechanical equipment may be needed, such as grinders. Typically, this step would be accomplished during a Level II cleaning. Level III inspections of steel elements often include ultrasonic thickness or caliper measurements and voltage potential measurements (Fig. 12-4). Ultrasonic thickness measurements identify remaining steel thickness, within a specific location, by measuring the return of ultrasonic waves in comparison to a calibrated source.

Because of the corrosive nature of the marine environment, coating and cathodic protection should be routinely monitored. Otherwise, the structure may prematurely deteriorate because of corrosion.

Marine environments are typically broken into five zones (atmospheric, splash, tidal, submerged, and subsurface). The highest corrosion rates are typically observed within the splash zone, followed closely by the interface between the tidal and submerged zones. Inspectors should have an understanding that steel corrosion is

Fig. 12-3. Concrete pile with major cracks and rust staining caused by corroded reinforcing steel

Source: Photo courtesy of Appledore Marine Engineering, LLC

Fig. 12-4. Inspector evaluating steel H-pile within high corrosion zone near low water

Source: Photo courtesy of Appledore Marine Engineering, LLC

most concentrated in areas with an abundance of moisture and oxygen, as well as areas with a change in electrical potential from differing materials or environments. Refer to Section 11.2 for a general description of deterioration modes. Fig. 11-2 presents a vertical profile of the relative loss of metal thickness by zones.

To monitor the relative loss of steel section, thickness measurements are often taken at four elevations: mudline, midwater, low water, and splash zone. It is also recommended that measurements be collected vertically at approximately 1-ft intervals over the full height of the element, commonly referred to as a *profile*. It is good practice for profile measurements to extend below the mudline because this can be an area of advanced corrosion if there is soil erosion and high current. By completing a profile, the inspectors can calibrate where the zones of advanced deterioration are located for future measurements, typically near low water and within the splash zone.

Other Level III inspection methods for steel elements may include coupon sampling or testing, coating thickness measurements, and voltage potential measurements. Coating thickness measurements provide insight into the expected durability and service life of the coating system.

During a Level I inspection of timber elements, gross defects often consist of split and broken members, marine borer deterioration, fungal decay, and corroded connections. Level II inspection typically reveals hidden marine borer deterioration and corroded connections. Level III investigations for timber elements often include

Fig. 12-5. Inspector completing moisture measurement on timber deck

Source: Photo courtesy of Appledore Marine Engineering, LLC

moisture measurements, shown in Fig. 12-5, and core samples, which allow for the evaluation of internal timber and preservative treatment depths.

Mooring hardware investigations should focus on connections, base, and fitting. An inspector must consider the structural load path when evaluating the condition of mooring hardware. For example, a poor connection may result in complete failure of a fitting that otherwise has no notable defects. The inspector should evaluate the connections to document any yielding of the steel, missing nuts, and corrosion. Investigation of the base should document stress cracks and whether the fitting is fully bearing on the base. Inspection of the fitting should document line wear, corrosion, and section loss. Many times, the interface between the base and fitting can exhibit advanced section loss. If this section loss becomes too advanced, failure of the fitting under load can result (Fig. 12-6).

A fender system is used to protect a structure and vessel during mooring and berthing. Often fender systems are constructed of sacrificial elements consisting of concrete, steel, timber, and composites. The investigation often consists of Level I with limited, more in-depth Level II and III investigation because of their sacrificial nature.

Waterfront utilities are routinely located along the face of a structure or hung beneath it, within areas that can often experience significant damage because of corrosion and impact. This damage often goes unnoticed and should not be overlooked during inspection because of the critical role utilities play in sustaining operations. The inspection should focus on the utility lines, hangers, stands, and connections,

Fig. 12-6. Steel pipe bollard that failed because of section loss within high corrosion zone

Source: Photo courtesy of Appledore Marine Engineering, LLC

noting corrosion and broken elements. Additional investigations should also be conducted to determine operational loads and to verify that the utilities meet code.

Retaining structures, including bulkheads and riprap, often contain structural slabs-on-grade, cast shoreside of the structure. Because of varying construction and design methods, the potential exists for material to wash out behind the retaining structure and go unnoticed until a large void is formed. During the waterfront inspection, if slabs-on-grade exist, the inspector should sound the structure as well as note cracks and settlement. Exploratory borings and ground-penetrating radar can be used to further examine areas suspected of having loss of material below a slab-on-grade.

12.3 Element and Facility Condition Assessments

A condition assessment provides the owner with a snapshot or projection of element, structure, and facility conditions. Condition assessments are based on engineering judgment, data collected in the field, and advanced material testing and evaluation. It is critical to understand the original structural capacity and existing levels of deterioration when assessing the condition of an element within a structure. With this information, an engineer can evaluate whether a structure is safe to operate as it was intended, if it needs to operate at a reduced capacity, or if it needs repair and/or replacement.

Original element capacity. Archive drawings and calculations are valuable resources for identifying original element strengths. If these details are unavailable, material testing can determine properties for capacity evaluation.

Original structural capacity. Archive drawings typically provide design criteria that state a structure's original allowable loading. If drawings are unavailable, evaluated element capacity, combined with structural analysis, can determine the original structural capacity.

Levels of deterioration. The amount and location of deterioration can significantly reduce an element's structural capacity. By factoring in structural redundancy or overdesign, compared to loss of structural capacity, repairs, load restrictions, or replacement may be required.

AASHTO (2010) states that knowledge of material vintage properties combined with experienced engineering judgment can provide estimates of material strength properties; however, collecting data and samples with follow-up testing may allow the use of higher stress values. Through data collection and advanced testing of these materials, one can

- Determine root causes of deterioration,
- Reveal hidden deterioration,
- Understand rates of deterioration,
- Determine original strength and durability,
- Determine existing strength, and
- Provide valuable design information that may be unknown if design and construction records are not available.

There are numerous data collection and advanced testing techniques for each material. The following examples provide common techniques for each of the three prominent materials used in waterfront construction.

Concrete. Petrographic analyses performed on concrete core samples provide a closer look at the concrete matrix. The analysis is performed by cutting a slice of the sample and examining it under a microscope in accordance with ASTM C856 (2014). Petrographic examination can identify common deficiencies in concrete including alkali–silica reaction (ASR), a chemical reaction between alkaline cement and reactive silica in aggregate, which causes destructive expansion of gel. Fig. 12-7 shows a concrete sample from a petrographic analysis that identifies ASR gel. Understanding the root cause of this premature deterioration allows for a proper repair to address the ASR and restore the structural capacity of the structure.

Steel. Ultrasonic thickness measurements are helpful when evaluating individual steel elements; these measurements may be extrapolated to model complete structures. By measuring the remaining web and flange thicknesses of corroded H-pile elements, existing geometric properties such as area, section modulus, and radius of gyration can be used for strength and serviceability calculations. In addition, comparing original versus existing steel thickness, combined with other geographical factors, can be used when evaluating local corrosion rates and projecting future load reductions or restrictions.

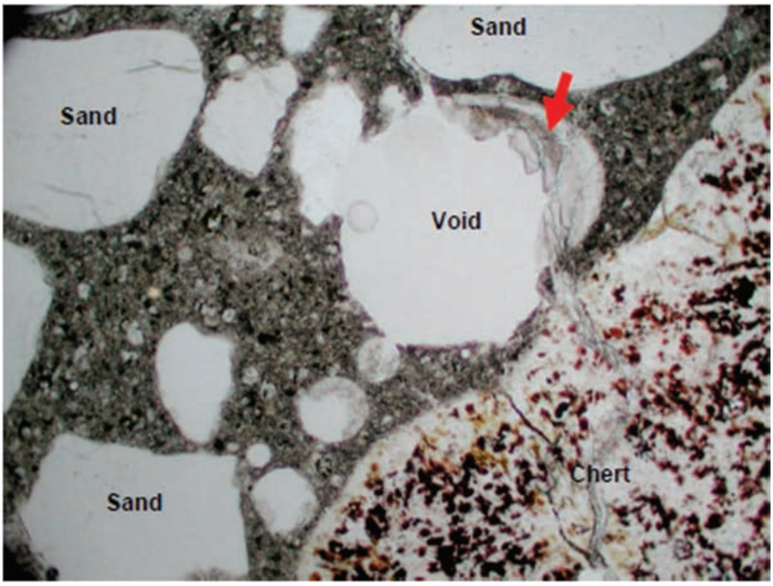

Fig. 12-7. Petrographic examination of a concrete sample exhibiting ASR

Source: Photo courtesy of Appledore Marine Engineering, LLC

Timber. Timber increment borer or drilling results require hands-on experience to provide meaningful interpretations and data. For the increment borer technique, the color and smell of extracted timber reveal quality and preservative treatment depth. Generally, a lighter coloring indicates sound timber. For the drilling technique, resistance of penetration, in combination with the color of chips, reveals the quality of timber. Sound timber is equated to a higher resistance of penetration with light-colored chips. A sudden lack of resistance is a tell-tale sign that internal deterioration is present. Fig. 12-8 depicts a diver drilling into a timber pile. The results of these techniques can be used to evaluate the element's remaining strength and provide insights regarding susceptibility of future deterioration. Larger diameter core samples may be taken for more in-depth assessment of marine borer presence and in-place timber strength.

Many references provide standardized recommendations for condition assessment. ASCE and the U.S. Navy have created useful guides that provide both qualitative and quantitative recommendations within a tiered rating system. Individual elements and entire facilities are categorized into the rating that best describes its condition.

It is important to understand which defects are minor versus those defects that compromise structural integrity. Hairline map cracking on top of a concrete deck, likely caused by thermal shrinkage, may appear substantial to the owner/user; however, it may not warrant repair or reduce capacity and therefore may have minimal effect on the deck's condition. Prestressed concrete support piles with

Fig. 12-8. Engineer-diver drilling into a timber pile

Source: Photo courtesy of Appledore Marine Engineering, LLC

exposed and partially failed prestressing strands significantly reduce the element's capacity and are likely to result in load restrictions. Fig. 12-9 shows a severely deteriorated concrete pile; however, because of the adjacent redundant pile, structural evaluation indicated that no load restrictions were required for the structure.

Isolated broken fender piles do not significantly compromise the integrity of a fender system; however, several broken piles in a row are likely to result in operational restrictions. Structures with widespread deterioration that have excess capacity and redundancy have a more favorable condition rating than a structure that has localized severe deterioration without redundancy. Ultimately, engineering judgment must be used to provide appropriate and meaningful assessments for each unique and complex facility.

12.4 Documentation and Reporting

During a waterfront facility inspection, it is critical to properly record notes in a clear, consistent, and concise manner. When data are postprocessed, any discrepancy or ambiguity in the documented conditions can lead to errors. The first step in clear note taking is to develop a standardized system through which notes can be correlated to the structure. Often, this is achieved by labeling pile bents and rows, in a numeric and alphabetic fashion, to create a gridlike pattern. Usually this pattern

Fig. 12-9. Severely deteriorated concrete pile; note exposed and corroded steel reinforcement

Source: Photo courtesy of Appledore Marine Engineering, LLC

has been established by the prior inspection or construction documents. For consistency, it is best to use existing labeling conventions. For some structures, stationing is most effective, for example, bulkheads or seawalls.

Uniformat II Elemental Classification is a method developed by the U.S. Department of Commerce that assigns unique identification numbers to each construction element (Charette and Marshall 1999). Using this system helps identify and organize the inspected elements, both during inspection and reporting. It also helps ensure that consistent data are collected from one inspection to the next.

Given the importance of documentation and advancements in technology, the industry is moving toward electronic documentation. There are numerous benefits, including the use of database software, wireless cloud-based storage media, and more efficient postprocessing. Commercially available tablets combine these technologies into a streamlined handheld unit (Fig. 12-10).

It is equally as important to collect visual documentation, including both videos and photographs. For many conditions, there is simply no substitute for a photograph. It is important to collect photographs of damages or defects, photographs that represent the overall conditions, and overview shots of the entire structure.

Underwater photographs present a series of challenges, given the lack of ambient light, currents that may exist, limited visibility, and suspended sediments in the water column. The use of a clear-water box can help in locations of poor visibility. A *clear-water box* is a customized translucent device that is filled with fresh

Fig. 12-10. Engineer-diver using a tablet to document conditions during a waterfront inspection

Source: Photo courtesy of Appledore Marine Engineering, LLC

deionized water. One example is shown in Fig. 12-11. This clean water displaces the ambient water and provides a better medium for underwater photography.

There are several key features to effective photographs or videos that need to be considered during an inspection. These include the following:

- A well-composed central point of interest;
- Proper exposure and contrast given the lighting environment;
- The use of a ruler, tape measure, or other form of dimensional scale (where possible);
- A locational reference (e.g., Pile 12:A); and
- Removal of marine growth for underwater photographs.

After the inspection, the next step is to create a deliverable that sufficiently captures the results of the specific inspection in a coherent manner. In most cases, the results of the inspection are summarized in report format. The contents of the report, as well as the format, vary based on the scope of the inspection, the complexity of the structure, and the specific needs of the owner (ASCE/COPRI 2015).

Reports may vary from a brief letter summarizing the inspection of a small structure to an in-depth and comprehensive report of several facilities at a major site. Each report, regardless of format or length, should include the following:

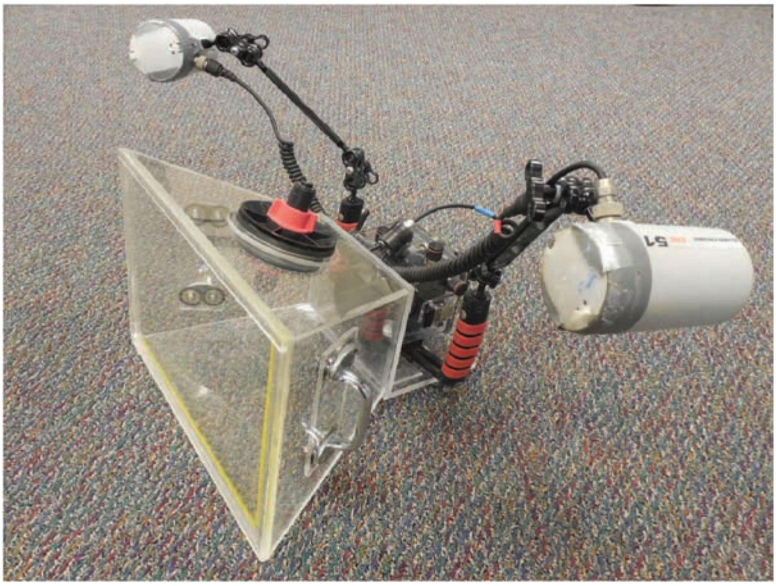

Fig. 12-11. A clear-water box is used to take underwater photographs in poor visibility

Source: Photo courtesy of Appledore Marine Engineering, LLC

- A statement of the scope of inspection or services provided,
- A statement of why the inspection was provided and/or the intent of the inspection,
- A summary of the conditions found during the inspection, and
- Sufficient representative visual documentation (photograph or video) of the inspection.

Depending on the scope of the inspection, the following may also be required:

- An engineering evaluation and assessment, including any assumptions made by engineering judgment;
- Recommendations made based on the results of the inspection, often prioritized based on the severity of the condition, the effect on operations, and the need for repair or subsequent design level or special inspection;
- Estimated cost of recommended repairs or studies to help the owner develop future budgets (estimates are typically developed assuming that each priority of repair is to be performed as one project, with consideration given for mobilization/demobilization, engineering fees, contractor overhead and profit, and other contingencies);
- Representative figures, including location map, vicinity plan, overall site plan, above-deck and under-deck plans, as well as typical sections and elevations; and

- Appendices providing raw data, either tabulated or from field notes. The results of any specialized testing or advanced inspection methods would be included as appendices.

The writing style of these reports should directly reflect the expertise of the inspector. Therefore, utmost importance should be placed on proper grammar, punctuation, and style. Tables, lists, and charts are preferred over long sentences and paragraphs. Whenever possible, use the "active voice" and do not use unnecessary qualifying clauses that detract from the subject of the sentence. The report should be consistent in the use of abbreviations, spelling out integers, and expressing measurements. There are several popular references available that provide guidance on technical writing style, including those published by the U.S. Government Printing Office (2008), Strunk and White (1999), and Han-Padron Associates (1999).

In addition to written reports, there are other methods of summarizing and presenting inspection data. One approach is the population of an asset management database. This is usually a predeveloped database, customized for each enterprise's needs, that tracks specific metrics for a given waterfront facility. The data are populated and updated in conjunction with each inspection. The database allows owners to manipulate, organize, and sort the data to help prioritize future repairs as part of an effective lifecycle management program. Building information modeling, or BIM, is another new trend used in lifecycle management programs that can be populated from data collected during waterfront inspections. This type of modeling entails using a 3D modeling software to store information on an element level. A model is created on the basis of a structure's Uniformat II elements. Each element in the model can be populated with data specific to that element.

These newer media for presenting or tracking data do not necessarily replace a physical report. It is simply an alternative way for owners to use the information collected during an inspection as part of lifecycle and asset management programs.

12.5 Lifecycle and Asset Management

Four phases make up the lifecycle of a structure: planning and design, construction, operation, and removal. To get the longest possible service life from a structure, a lifecycle management (LCM) program is important. It is often the case that the major aspects of infrastructure maintenance are typically looked at only after a problem has been identified. Instead, these financial, technical, environmental, and safety aspects should be evaluated during each phase of the structure's life. This is referred to as whole-life costing (WLC) analysis. These concepts are further elaborated upon in two PIANC publications; *Life Cycle Management of Port Structures: General Principles* (PIANC 1998) and *Life Cycle Management of Port Structures, Recommended Practice for Implementation* (PIANC 2008). Lifecycle management is an approach to the management of infrastructure construction to achieve cost-effective functionality and quality. The goal of the program is to achieve minimum costs throughout the life of the structure.

Asset management is an important part of an LCM program, an essential tool when tasked with managing a structure throughout its service life. It is used to forecast the short-term and long-term repairs, as well as anticipated future repair costs for a structure. These tools may vary from simple spreadsheet databases that can be set up to manage an individual structure or small facility to detailed software products that can be used to manage entire ports. No matter the size of the project, the process is the same; the inspection and analysis data that are collected during the routine and special inspection processes are input into the management tool and tracked to determine multiple elements, including deterioration rates, repair costs, and level of importance. These tools can assist in prioritizing and projecting repairs to ensure that the structure's service life meets or exceeds its design life.

Electronic data collection can be incorporated into an asset management tool. Through tailored database tools, the inspection data can be directly populated into the asset management tool, reducing errors while allowing more effective and efficient use of the collected data. A database can be used to prioritize repairs. These databases should be set up to include all structures within a facility; each structure should have its own section. The structure can also be broken down even further to identify the elements of the substructure, superstructure, and critical appurtenances. The rating system, including short- and long-term recommended repairs and cost estimates for those repairs, can all be populated into the database. Queries and reports can be input into the database to provide a wealth of information, including next routine inspection dates, priority repairs, future repair costs based on the year the repairs are required, and the estimated remaining service life. Together these tools enable a facility owner to develop a more robust maintenance repair program that can prolong the service life of the structure.

Concrete service life prediction models based on finite element analysis techniques, such as STADIUM (STI 2011) and Life-365 (Life-365 2014), for example, can model the course of concrete properties and degradation over the structure life to assist in managing it. They have the capability of tracking rates of deterioration to provide detailed predictions on the future of a structure. For example, a pervasive mode of deterioration of concrete in the marine environment is chloride ion ingress. By taking concrete cores and measuring the levels of chlorides in the concrete, a finite element model can be calibrated to predict the rate at which the chlorides will reach the reinforcing steel and, in turn, cause deterioration of the steel and surrounding concrete. The finite element model takes into account the chemical composition of the concrete, the geographical location, depth of reinforcing steel, overall structure dimensions, and the levels of chlorides already present at varying depths in the concrete. The model can produce a graph that predicts the number of years it will take for the chlorides to cause deterioration of the reinforcing steel; it also predicts the depth of the repair to stop the ingress of chlorides. Service life prediction of concrete structures is addressed in detail in ACI 365.1R (ACI 2000).

Management tools can also assist in understanding the lifecycle costs necessary for maintaining a structure. Fig. 12-12 illustrates the importance of this concept. Bulkheads can be constructed from a variety of materials, each with varying initial costs. Initial investigations may indicate that a timber bulkhead would be the least expensive option. However, considering the cost associated with necessary maintenance to achieve the intended service life, the costs of a timber bulkhead may not be the least expensive over its lifecycle; see Fig. 12-12. Additionally, slight changes in configuration, such as installing a concrete cap that extends below mean lower low water (MLLW), results in significantly lower maintenance costs.

Lifecycle costs of a structure not only include the initial construction costs, but also take into consideration the inspection, maintenance, and major repair projects that are required for the structure throughout its life. Considering the overall lifecycle cost, it could be determined that a steel or concrete structure would be

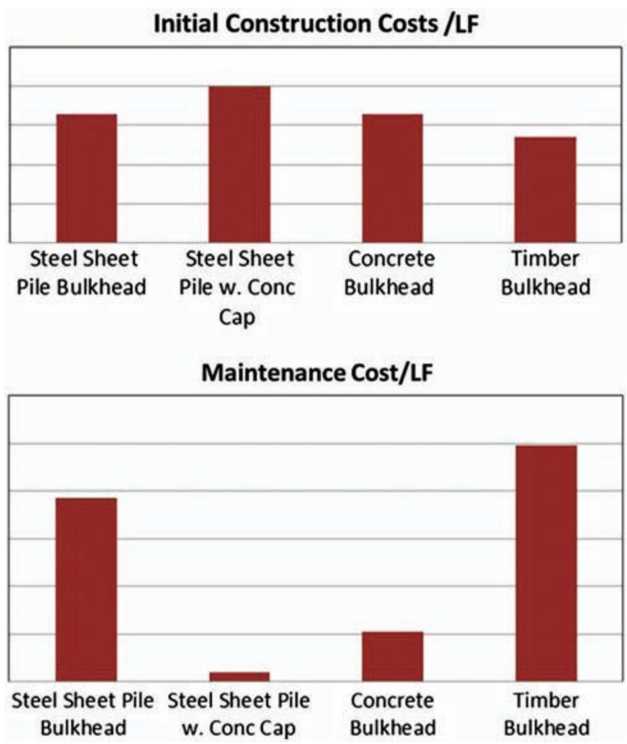

Fig. 12-12. Comparison of initial construction costs per lineal foot and maintenance costs per lineal foot for different bulkhead configurations

Source: Courtesy of Appledore Marine Engineering, LLC a better choice. However, without an effective inspection and maintenance program to manage the asset throughout its life, the full picture would not be apparent. Voogt et al. (2014) describe an aggressive waterfront asset management program for the port of Rotterdam, Netherlands, which was developed in partnership with an asset management tool called KMS (the Dutch abbreviation for quay wall modeling system). KMS uses the results of deterioration models for steel and concrete and compares it with the "end of contract" date to identify and rank the risks that endanger the functionalities of the structure and uses the business value of a quay wall to clarify its maintenance priorities. Paulsen et al. (2013) describe an asset management program developed for the port of Tacoma, Washington, that integrates the full range of port activities related to the built and natural infrastructure of the port, including marine facility maintenance management, lifecycle analysis, prioritized condition assessments, GIS, a facility information database, port business information, and decision making.

Acknowledgments

This chapter was developed by the staff of Appledore Marine Engineering, LLC, including Stephanie N. Baratt, P.E.; Noah J. Elwood, P.E., President; Linn B. Lebel, P.E.; Eric T. Levesque, P.E.; David L. Marcotte, P.E.; Daniel B. Robbins, P.E.; Matthew L. Teeden, P.E.; and Kyle M. Vandemoer, EIT.

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ASCE. (2001) *Underwater investigations: Standard practice manual*, K. M. Childs, Jr., ed., ASCE, Reston, VA.

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Paulsen, L., et al. (2013). "Asset management program development and implementation for the port of Tacoma, Tacoma, WA." *ASCE Ports 2013*, Seattle, WA.

PIANC. (1998). "Life cycle management of port structures: General principles." International Navigation Association, PIANC, Brussels, Belgium.

PIANC. (2008). "Life cycle management of port structures, recommended practice for implementation." *MarCom Rep. 103, World Association of Waterborne Transport Infrastructures, PIANC, Brussels, Belgium.*

STI (Simco Technologies Inc). (2011). *STADIUM users guide, v 2.99*, Québec.

Strunk, W., and White, E. B. (1999). *The elements of style*, Pearson Education, London.

U.S. Government Printing Office. (2008). *Style manual: An official guide to the form and style of federal government printing*, Washington, DC.

Voogt, H., et al. (2014). "Proactive and predictive waterfront asset management—Port of Rotterdam experience." *World Congress*, PIANC, Brussels, Belgium.

Conversion Factors

Customary to Metric Units

U.S. (Customary)	SI (Metric)
1 inch (in.)	2.54 centimeters (cm)
1 foot (ft)	0.3048 meter (m)
1 yard (yd)	0.9144 meter (m)
1 statute mile	1.609 kilometers (km)
1 square inch (sq in.)	6.45 square centimeters (cm^2)
1 square foot (sq ft)	0.093 square meter (m^2)
1 cubic inch (cu in.)	16.39 cubic centimeters (cc)
1 cubic foot (cu ft)	0.0283 cubic meter (m^3)
1 foot per second (ft/s)	0.3048 meters per second (m/s)
1 pound (lb)	0.453 kilogram (kg)
1 pound (lb) (force)	4.448 newtons (N)
1 pound-foot (lb-ft)	1.356 newton-meter (N-m = Joule [J])
1 ton (2,000 lb)	0.907 metric ton (m.t.)
1 pound per square inch (psi)	6.89 kilo-newtons per square meter (kN/m^2)
1 pound per square foot (psf)	47.88 Newtons per square meter (N/m^2 = Pascal [Pa])
1 pound per cubic foot (pcf)	16.02 kilograms per cubic meter (kg/m^3)
1 kilo-pound (kip) (force)	4.448 kilo-newtons (kN)
1 kip-foot (kip-ft)	1.356 kilo-newton-meters (kN-m)
1 kip per foot (kip/ft)	14.59 kilo-newtons per meter (kN/m)
1 horsepower (HP)	745.7 watts (W = Joule/second)

Nautical Units

1 nautical mile = 1.151 statute miles = 6076.1 ft = 1.852 km
1 knot = 1.151 miles per hour = 1.688 ft/s = 0.515 m/s
1 fathom = 6.0 ft = 1.829 m
1 long ton = 1.12 short tons = 2,240 lb = 1.016 metric tons

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Selected Information Sources: Journals, Periodicals, and Conference Proceedings

Technical Journals

Journal of Waterway, Port, Coastal, and Ocean Engineering, American Society of Civil Engineers, Reston, VA (bimonthly).

Coastal Engineering: An International Journal for Coastal, Harbour and Offshore Engineers, Elsevier Scientific Publishing Co., Amsterdam, Netherlands (quarterly).

Coastal Engineering Journal, World Scientific Publishing Co., Singapore.

On Course: PIANC Magazine, The World Association for Waterborne Transport Infrastructure, a.k.a. PIANC, the Permanent International Association of Navigation Congresses, Brussels, Belgium (quarterly).

Terra et Aqua, International Association of Dredging Companies, The Hague, Netherlands (quarterly).

Marine Structures, published in association with the International Ship and Offshore Structures Congress by Elsevier B.V., Essex, U.K.

Periodicals

Ports and Harbours Magazine, IHS Journal of the International Association of Ports and Harbours, IAPH (bimonthly).

IHS Dredging and Port Construction, published by IHS, Northampton, U.K. (monthly).

Port Technology International, Maritime Information Services, Ltd., London (a few per year).

Offshore, PennWell Publishing Co., Tulsa, OK (monthly).

Sea Technology, Compass Publications Inc., Arlington, VA (monthly).

World Dredging, Mining and Construction, Symcon Publishing Co., Irvine, CA (bimonthly).

Cargo Systems International, Journal of the International Cargo Handling and Coordination Association, London (monthly).

World Port Development, Berkshire, U.K. (monthly).

World Wide Shipping, WWS, Blauvelt, NY (eight issues per year).

World Port and Harbor Abstracts, British Harbour Research Association, London (quarterly).

Conference Proceedings

Selected regularly held conferences are listed; numerous other specialty conferences of interest are held sporadically by ASCE and others.

PORTS conferences, American Society of Civil Engineers/Coasts, Oceans, Ports and Rivers Institute (ASCE/COPRI); held every 3 years since 1977.

Civil Engineering in the Oceans (ASCE); six conferences held, in 1968, 1969, 1975, 1979, 1992, and 2004.

Coasts, Marine Structures and Breakwaters (ICE), Institution of Civil Engineers, London, held periodically.

Coastal Structures Conferences (ASCE); held intermittently, 1979, 1999, and 2003.

International Conference on Coastal Engineering (ICCE) (ASCE); first conference held in 1950, 35th conference planned for 2016, formerly Coastal Engineering Conference (CEC).

International Conference on Coastal and Port Engineering in Developing Countries (COPEDEC); 9th conference to be held 2016.

International Conference on Coasts, Ports and Marine Structures (ICOPMAS), first conference held 1990 and 11th conference held 2014.

International Navigation Congress, PIANC World Congress, (INA)/PIANC; held intermittently since 1885, 33rd congress held in 2014.

Port and Terminal Technology, MCI Media, Ltd., White Waltham, U.K., 8th International Conference and Exhibition held in Charleston, SC, 2016 (annual).

World Ports Conference of the International Association of Ports and Harbours (IAPH); held intermittently, 16th conference in 1989.

Conference on Ocean Engineering under Arctic Conditions (POAC); first conference in 1971, 7th conference in 1983.

Offshore Technology Conference; held annually in Houston since 1969.

World Maritime Technology Conference (WMTC)/SNAME Maritime Convention, Society of Naval Architects and Marine Engineers, Alexandria, VA (triennial)

World Dredging Conference (WODCON), World Organization of Dredging Associations; held every 3 years since 1967.

Docks and Marinas Conference, University of Wisconsin Continuing Education Program, Madison, WI, held annually since 1974.

Sources of Vessel Data

American Ship Review, Professional Mariner, Portland, ME (annual).

Marine Technology, Society of Naval Architects and Marine Engineers (SNAME), Alexandria, VA (quarterly).

Marine Log, Simmons-Boardman Publishing Corp., New York (monthly).

Maritime Reporter and Engineering News, Maritime Activity Reports, Inc., New York (monthly).

Shipping Today and Yesterday, HPC Publishing, East Sussex, U.K. (monthly).

Shipping World and Shipbuilder, Marine Publications International, Ltd., London (monthly).

Ship and Boat International, Metal Bulletin Journals, Ltd., London (10 issues per year).

Riviera Maritime Media, Ltd., publishes quarterly journals of ship technology for specific ship types, including tankers, LNG, passenger and ferries, Enfield, U.K.

Work Boat, Diversified Business Communications and Publications, Portland, ME (monthly).

Yachts International magazine, Ft. Lauderdale, FL (monthly).

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Internet Sources and Selected Websites

Design Guidance, Regulations, and Official Government Sites

ADA Accessibility Guidelines (ADAAG)	http://www.access-board.gov
American Meteorological Society (AMS)	http://www.ametsoc.org
Americans with Disabilities Act (ADA)	http://www.ada.gov
California Department of Boating and Waterways (DBW)	http://www.dbw.ca.gov/TechDocs
California State Lands Commission (SLC)	http://www.slc.ca.gov
Defense Technical Information Center	http://www.dtic.mil/dtic
Federal Emergency Management (FEMA)	http://www.fema.gov
Intergovernmental Panel on Climate Change (IPCC)	http://www.ipcc.ch
National Institute of Standards and Technology (NIST)	http://www.nist.gov/
National Transportation Safety Board (NTSB)	http://www.ntsb.gov
Naval Facilities Engineering Command (NAVFAC)	http://www.navy.mil/local/navfachq
Occupational Safety and Health Administration (OSHA)	http://www.osha.gov
Pile Buck (reprints of out-of-print design guide publications)	http://www.pilebuckinternational.com
University National Oceanographic Laboratory System (UNOLS)	http://www.unols.org
U.S. Coast Guard Marine Safety Center (MSC)	http://www.uscg.mil/hq/msc
U.S. Department of Homeland Security (DHS)	http://www.dhs.gov
U.S. Environmental Protection Agency (EPA)	http://www.epa.gov
U.S. Geological Survey (USGS)	http://www.usgs.gov
U.S. Government Publication Office, Code of Federal Regulations (CFR)	http://www.gpo.gov/
U.S. Government Publication Office, Federal Digital System (FDSYS)	http://www.gpo.gov/fdsys
USACE Coastal and Hydraulics Engineering Technical Notes (CHETN)	http://chl.erdc.usace.army.mil/chetn
USACE Coastal and Hydraulics Laboratory (CHL)	http://chl.erdc.usace.army.mil

USACE Coastal Engineering Manual (CEM)	http://chl.erdc.usace.army.mil/cem
USACE Engineer Research and Development Center (ERDC)	http://www.erdc.usace.army.mil
USDA Forest Service, Forest Products Laboratory (FPL)	http://www.fpl.fs.fed.us
Whole Building Design Guide (WBDG) (Unified Facilities Criteria and other public domain design documents can be downloaded from this important site.)	http://www.wbdg.org
World Association for Waterborne Transport Infrastructure (PIANC)	http://www.pianc.org
World Meteorological Organization (WMO)	http://www.wmo.int

Professional Organizations

Aluminum Association (AA)	http://www.aluminum.org
American Association of Port Authorities (AAPA)	http://www.aapa-ports.org/
American Concrete Institute (ACI)	http://www.concrete.org
American Forest & Paper Association (AFPA)	http://www.afandpa.org
American Galvanizers Association	http://www.galvanizeit.org
American Institute of Steel Construction (AISC)	http://www.aisc.org
American Institute of Timber Construction (AITC)	http://www.aitc-glulam.org
American National Standards Institute (ANSI)	http://www.ansi.org
American Petroleum Institute (API)	http://www.api.org
American Society for Metals (ASM)	http://www.asminternational.org
American Society of Civil Engineers (ASCE)	http://www.asce.org
American Society of Mechanical Engineers (ASME)	http://www.asme.org
American Welding Society (AWS)	http://www.aws.org
American Wood Council (AWC)	http://www.awc.org
American Wood Protection Association (AWPA)	http://www.awpa.com
Asphalt Institute (AI)	http://www.asphaltinstitute.org
Associated General Contractors of America (AGC)	https://www.agc.org
Association of Marina Industries (AMI)	https://marinaassociation.org
ASTM International (ASTM)	http://www.astm.org
Concrete Reinforcing Steel Institute (CRSI)	http://www.crsi.org
Construction Criteria Base (CCB)	http://www.wbdg.org/ccb
Construction Industry Research & Information Association (CIRIA)	http://www.ciria.org
Construction Specifications Institute (CSI)	http://www.csinet.org

Global Marina Institute (GMI)	http://globalmarinainstitute.net
Hoist Manufacturers Institute (HMI)	http://www.mhi.org
International Association for Hydro-Environment Engineering and Research	http://www.iahr.org
International Association of Lighthouse Authorities (IALA)	http://www.iala-aism.org
International Association of Ports and Harbors (IAPH)	http://www.iaphworldports.org
International Boat Industry (IBI)	http://www.ibinews.com
International Cargo Handling and Coordination Association (ICHCA)	http://www.ichca.com
International Code Council (ICC)	http://www.iccsafe.org
International Concrete Repair Institute (ICRI)	http://www.icri.org
International Maritime Organization (IMO)	http://www.imo.org
National Association of Corrosion Engineers (NACE)	http://www.nace.org/home.aspx
National Fire Protection Association (NFPA)	http://www.nfpa.org
National Institute of Building Sciences (NIBS)	http://www.nibs.org
Oil Companies International Marine Forum (OCIMF)	http://www.ocimf.org
Plastics Pipe Institute (PPI)	http://www.plasticpipe.org
Portland Cement Association (PCA)	http://www.cement.org
Precast/Prestressed Concrete Institute (PCI)	http://www.pci.org
Royal Institution of Naval Architects (RINA)	http://www.rina.org.uk
Society of International Gas Tanker and Terminal Operators (SIGTTO)	http://www.sigtto.org/
Society of Naval Architects and Marine Engineers (SNAME)	http://www.sname.org
Southern Forest Products Association (SFPA)	http://www.sfpa.org/
Southern Pine Council (SPC)	http://www.southernpine.com
Southern Pine Inspection Bureau (SPIB)	http://www.spib.org
Steel Structures Painting Council (SSPC)	http://www.sspc.org
Timber Piling Council	http://timberpilingcouncil.org/
United Nations Conference on Trade and Development (UNCTAD)	http://unctad.org/en/Pages/Home.aspx
United Nations, Department of Economic and Social Affairs	https://www.un.org/development/desa/en/
The Waterfront Center (TWC)	http://www.waterfrontcenter.org
Western Wood Preservers Institute (WWPI)	http://www.wwpinstitute.org
Wire Reinforcement Institute (WRI)	http://www.wirereinforcementinstitute.org
World Association for Waterborne Transport Infrastructure (PIANC)	http://www.pianc.org
Woven Wire Products Association (WWPA)	http://www.wovenwire.org
Yacht Harbor Association (TYHA)	http://www.tyha.co.uk

Tide and Water Level Data

NOAA Coastal Services Center	https://www.climate.gov
NOAA National Geodetic Survey (NGS)	http://geodesy.noaa.gov
NOAA National Ocean Service (NOS)	http://oceanservice.noaa.gov
NOAA NOS Tides and Currents	tidesandcurrents.noaa.gov
NOAA NOS Tides Online	http://tidesonline.nos.noaa.gov
USACE and NOAA Sea Level Change Curve Calculator	http://www.corpsclimate.us/ccaceslcurves.cfm

Wind, Wave, and Environmental Data

National Renewable Energy Laboratory (NREL)—Wind Data	http://www.nrel.gov/gis/data_wind.html
NOAA National Centers for Environmental Information (NCEI)	http://www.ncdc.noaa.gov
NOAA National Data Buoy Center (NDBC)	http://www.ndbc.noaa.gov
NOAA National Oceanographic Data Center (NODC)	http://www.nodc.noaa.gov
NOAA National Weather Service (NWS)	http://www.weather.gov
NWS Homepage	http://www.nws.noaa.gov
NWS International Tsunami Information Center (ITIC)	http://www.prh.noaa.gov/itic
NWS Marine Forecasts	http://www.nws.noaa.gov/om/marine/home.htm
NWS National Centers for Environmental Prediction (NCEP)	http://www.ncep.noaa.gov
NWS National Hurricane Center (NHC)	http://www.nhc.noaa.gov
NWS NCEP Central Operations (NCO)	http://www.nco.ncep.noaa.gov
NWS Storm Prediction Center (SPC)	http://www.spc.noaa.gov
Surfline (wave forecasts)	http://www.surfline.com/surf-forecasts
USACE Wave Information Studies (WIS)	http://wis.usace.army.mil
Weather Underground—Tropical weather	http://www.wunderground.com/tropical

Charts

National Geospatial-Intelligence Agency (NGA)	https://www.nga.mil/Pages/Default.aspx
NGA Digital Nautical Chart (DNC)	http://dnc.nga.mil
NOAA Office of Coast Survey (OCS)	http://www.nauticalcharts.noaa.gov

Dredging and Marine Construction

Bollard Load Testing, Ltd. (BLT)	bollardloadtest.com
Cavotec SA (automated mooring systems)	http://www.cavotec.com/
Dredging Contractors of America (DCA)	dredgingcontractors.org

International Association of Dredging Companies (IADC)	http://www.iadc-dredging.com
Pile Buck	http://www.pilebuckinternational.com/
Western Dredging Association (WEDA)	http://www.westerndredging.org
World Organization of Dredging Associations (WODA)	www.woda.org

Vessel Data

American Bureau of Shipping (ABS)	http://www.eagle.org
Clarkson Research Services Limited	http://www.crsl.com
CSL International (bulk carrier data)	http://www.cslships.com/en/csl-americas
Federation of American Scientists—U.S. Navy ships	http://www.fas.org/man/dod-101/sys/ship
Haze Gray & Underway—Naval history and photography	http://www.hazegray.org
IHS Maritime World Register of Ships	https://www.ihs.com/products/maritime-world-ship-register.html
International Maritime Organization (IMO)	http://www.imo.org
Lloyd's Register of Shipping	http://www.lr.org
Maritime Connector	http://maritime-connector.com
Naval ship information—Photographic history of the U.S. Navy	navsource.org
Naval Vessel Register	http://www.nvr.navy.mil
Ocean Tug & Barge Engineering	oceantugbarge.com
Q88 (tanker data)	q88.com
Riviera Maritime Media	rivieramm.com
Seatrade	http://www.seatrade.com
Shipping database	http://www.shippingdatabase.com
Steve's Maritime (ship type descriptions)	stevesmaritime.com
Tanker fleet	oceantankers.com
U.S. Maritime Administration (MARAD)	http://www.marad.dot.gov
University National Ocean Laboratory System (UNOLS)	unols.org
Vessel Finder	https://www.vesselfinder.com

Ports, Shipping, and Maritime News

American Association of Port Authorities (AAPA)	http://www.aapa-ports.org
Drewry Shipping Consultants, Ltd.	drewry.co.uk
Federal Maritime Commission (FMC)	http://www.fmc.gov
IHS Fairplay Ports News	ihsmaritime360.com
IHS Journal of Commerce, International Ports (JOC)	http://www.joc.com

International Association of Ports and Harbors (IAPH)	http://www.iaphworldports.org
Marine Exchange of Southern California	http://www.mxsocal.org
Maritime Today	http://www.maritimetoday.com
Port Technology International	http://www.porttechnology.org
U.S. Coast Guard (USCG)	https://www.uscg.mil
Workboat	workboat.com
World Port Development	worldportdevelopment.com

Dry Docks

American Bureau of Shipping (ABS)	http://www.eagle.org
Crandall Dry Dock Engineers, Inc.	http://www.crandalldrydock.com
DM Consulting	http://www.drydocktraining.com
Drydock Magazine	drydockmagazine.co.uk
Frank Shipbrokers LTDA	www.frankshipbrokers.com
GHS Port Reference Center	http://www.ghsport.com
Heger Dry Dock, Inc.	http://www.hegerdrydock.com
Marine Travelift, Inc.	http://www.marinetravelift.com
Pearlson Shiplift Corporation (Syncrolift history)	http://www.shiplift.com
Proteus Engineering	http://www.proteusengineering.com

Software

ANSYS/AQWA	ansys.com/Products/Structures/ANSYS-AQWA
FIXMOOR	https://www.wbdg.org/tools/fixmoor.php?a=1
FLAC	itascacg.com/software/flac
L-Pile	ensoftinc.com
Life-365	life-365.org
MIKE-21	mikepoweredbydhi.com/products/mike-21
OPTIMOOR	http://www.tensiontech.com/software/optimoor.html
PLAXIS	plaxis.nl
SPW911	http://www.pilebuckinternational.com/product/spw911-sheet-pile-design-software/
STADIUM	http://www.simcotechnologies.com/what-we-do/stadium-technology-portfolio/
TERMSIM	marin.nl
Veritech Enterprises, CEDAS/ includes ACES program	veritechinc.com

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About the Author

John W. Gaythwaite, P.E., is president of Maritime Engineering Consultants, Inc., in Manchester, MA. He is a registered professional engineer in several states with more than 40 years of experience specializing in the design of marine structures. He has served as a project manager and peer reviewer for the design of many marine civil engineering projects, including dry docks, shipyard and marine terminal facilities, small craft harbors, and coastal engineering works.

Gaythwaite is an active member of various professional technical committees and has appeared as a guest lecturer at several universities and at many professional meetings. He is a life member of the American Society of Civil Engineers, where he currently serves on the Ports and Harbors technical committee of the Coasts, Oceans, Ports, and Rivers Institute (COPRI). As a member of the Boston Society of Civil Engineers, he serves on the editorial board of the *Journal of Civil Engineering Practice*. He is a board-certified Diplomate in Coastal Engineering and in Port Engineering of the COPRI Academy of Coastal, Ocean, Port & Navigation Engineers (ACOPNE). Gaythwaite is also a longtime member of the Permanent International Association of Navigation Congresses (PIANC) and the Society of Naval Architects and Marine Engineers (SNAME). He is an experienced offshore sailor and holds a U.S. Coast Guard 100-ton masters license.

He is the editor of *Mooring of Ships to Piers and Wharves* (ASCE Manuals of Practice and Reports on Engineering Practice No. 129, 2014) and author of *The Marine Environment and Structural Design*. He has published technical papers on a diverse range of topics, including floating breakwaters, coastal zone construction and shore protection, waterfront rehabilitation, design of an offshore research platform, and arctic port structure restoration.

In 2010, John Gaythwaite received the prestigious ASCE/COPRI John G. Moffatt–Frank E. Nichol Harbor & Coastal Engineering Award, which cited his contributions to expanding the civil engineering community's knowledge of the marine environment, his production of standard texts used by the industry, and his dedication to the civil engineering profession.